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OZONE DISINFECTION OF WASTEWATERS USING COMPLETE MIX REACTORS

BY

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DEDICATION

This thesis is dedicated to: my fiancé, Sue; my parents, Leno and Inez; my brother, Dale; my grandparents, Fiore, Pietro, Rosina, and Maria; and my future in-laws, Bruce and Pam. Their encouragement and understanding ensured the successful completion of this project.

ABSTRACT

A pilot scale plant was established at the City of Edmonton's Goldbar wastewater treatment plant to examine the effectiveness of ozone as a wastewater disinfectant using a complete mix reactor. Tests were conducted to examine the ability of ozone to inactivate total and fecal coliforms and to determine the effect of wastewater and operating characteristics on the disinfection process. The main factors affecting the disinfection process were the ozone dose applied to, or the ozone utilized in the complete mix reactor. Other factors which demonstrated a significant effect on coliform survival include the wastewater BOD_5 and the ozone residual established in the wastewater.

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TABLE OF CONTENTS

Chapter			Page
I	INTRODUCTION		1
II	LITERATURE REVIEW		2
	A.	HISTORICAL DEVELOPMENT	2
	B.	EFFECTIVENESS OF OZONE VERSUS CHLORINE AS A DISINFECTANT	4
	C.	TOXICITY OF CHLORINATED AND OZONATED EFFLUENTS	5
	D.	CONTACT TIME REQUIREMENTS FOR OZONE AND CHLORINE DISINFECTION	16
	E.	CONSIDERATION OF DISINFECTION ALTERNATIVES	16
	F.	EFFECT OF WASTEWATER QUALITY ON OZONE DISINFECTION	18
	G.	OZONE MASS TRANSFER AND MIXING SYSTEMS	20
	H.	OZONE REQUIREMENTS FOR WASTEWATER DISINFECTION	24
	I.	WASTEWATER CHARACTERISTICS	24
III	PROJECT OBJECTIVES		29
IV	PROJECT COMPONENTS		30
	A.	REACTOR MODELLING	30
	B.	LABORATORY FACILITIES AND PROCEDURES	33
	C.	WASTEWATER ANALYSIS	41
	D.	APPLIED OZONE DOSE, UTILIZED OZONE DOSE AND OZONE RESIDUAL	41
	E.	DATA ANALYSIS	43
V	RESULTS		44
	A.	DYE TESTS	44

TABLE OF CONTENTS

Chapter			Page
	B.	CONTROL TESTS	44
	C.	DISINFECTION RESULTS	44
	D.	STATISTICAL ANALYSIS	78
VI	DISCUSSION		84
	A.	DYE TESTS	84
	B.	PRIMARY EFFLUENT CHARACTERISTICS	84
	C.	CONTROL TESTS	86
	D.	DISINFECTION TESTS AND PREDICTIVE RELATIONSHIPS	86
VII	CONCLUSIONS		95
VIII	RECOMMENDATIONS		96

* * *

BIBLIOGRAPHY		97
APPENDIX 1	DYE TEST DATA	103
APPENDIX 2	DISINFECTION TEST DATA	113

LIST OF TABLES

Table	Description	Page
1	VALUES OF THE LETHAL UNIT, Λ	6
2	OXIDATION POTENTIAL OF CHEMICAL DISINFECTANTS	7
3	SELECTED FIELD STUDIES DEMONSTRATING EFFECTS OF CHLORINATED SEWAGE ON AQUATIC LIFE	9
4	ACUTE TOXICITY OF RESIDUAL CHLORINE TO FRESHWATER FISH	10
5	CHRONIC TOXICITY OF RESIDUAL CHLORINE TO FRESHWATER ORGANISMS	11
6	TOXIC EFFECTS OF CHLORINATED EFFLUENTS ON MARINE INVERTEBRATES	12
7	CHLORINATED ORGANICS IN WASTEWATER EFFLUENTS	14
8	ADVANTAGES AND DISADVANTAGES OF OZONE MIXING SYSTEMS	25
9	COMPARISON OF OZONE DOSES REPORTED IN WATER AND WASTEWATER TREATMENT	27
10	WASTEWATER CHARACTERISTICS	28
11	CONTROL TEST RESULTS FOR TOTAL COLIFORMS	69
12	CONTROL TEST RESULTS FOR FECAL COLIFORMS	70
13	SIMPLE LINEAR REGRESSION EQUATIONS RELATING SURVIVAL RATIOS TO APPLIED OZONE DOSE, OZONE UTILIZED AND OZONE RESIDUALS	79
14	SUMMARY OF REGRESSION ANALYSIS FOR BACTERIAL SURVIVAL IN PRIMARY EFFLUENT	80
15	SUMMARY OF PREDICTIVE RELATIONSHIPS FOR ON-LINE USE	82
16	MULTIPLE LINEAR REGRESSION EQUATIONS WITH OZONE RESIDUAL AND OZONE UTILIZED AS DEPENDENT VARIABLES FOR A COMPLETE MIX REACTOR SYSTEM	83

LIST OF TABLES (cont'd)

Table	Description	Page
17	RANGE OF PRIMARY EFFLUENT CHARACTERISTICS IN THE STUDY	85
18	COMPARISON OF SIMPLE LINEAR REGRESSION EQUATIONS FOR COMPLETE MIX AND POROUS PLATE DIFFUSER SYSTEMS	89
19	COMPARISON OF MULTIPLE LINEAR REGRESSION EQUATIONS FOR COMPLETE MIX AND POROUS PLATE DIFFUSER SYSTEMS	91
20	COMPARISON OF PREDICTIVE RELATIONSHIPS FOR THE COMPLETE MIX AND POROUS PLATE DIFFUSER SYSTEMS	93

LIST OF FIGURES

Figure		Page
1	FILM THEORY OF GAS TRANSFER	22
2	COMPLETE MIX REACTOR	31
3	EXPONENTIAL DECAY OF COMPLETE MIX REACTOR WHEN SUBJECT TO A SLUG DYE INPUT	32
4	SCHEMATIC DIAGRAM OF THE DYE TESTING SYSTEM	34
5	SCHEMATIC OF OZONE PILOT PLANT	39
6-29	DYE DECAY CURVES FOR VARIOUS REACTOR OPERATING CONDITIONS	45 to 68
30	APPLIED OZONE DOSE VS. TOTAL COLIFORM SURVIVAL RATIO	71
31	OZONE UTILIZED VS. TOTAL COLIFORM SURVIVAL RATIO	72
32	OZONE RESIDUAL VS. TOTAL COLIFORM SURVIVAL RATIO	73
33	APPLIED OZONE DOSE VS. FECAL COLIFORM SURVIVAL RATIO	74
34	OZONE UTILIZED VS. FECAL COLIFORM SURVIVAL RATIO	75
35	OZONE RESIDUAL VS. FECAL COLIFORM SURVIVAL RATIO	76

LIST OF PHOTOGRAPHIC PLATES

Plate		Page
1	DYE TESTING SYSTEM	35
2	EXPERIMENTAL TRAILER	36
3	EXPERIMENTAL TRAILER	37
4	MICROBIOLOGY TRAILER	38

I INTRODUCTION

In recent years rapid population growth and improved standards of living have led to the discharge of wastewater effluents to recreational and drinking water supplies. As a result the transmission of waterborne diseases has become a perpetual problem.

The destruction of microorganisms through disinfection is generally considered to be the most effective method of preventing the spread of waterborne diseases. At the present time chlorine is the most widely used wastewater disinfectant. However chlorinated effluents are toxic to aquatic life and have been found to contain chlorinated organic compounds which are suspected carcinogens.

There are two methods available to achieve the desired level of microorganism inactivation and reduce effluent toxicity. The first is to reduce the toxicity of chlorinated effluents by following chlorination with dechlorination procedures. The second involves the utilization of an alternate disinfectant. Potential replacements for chlorine include ozone, ultraviolet light, chlorine dioxide, bromine and iodine.

Historically ozone has not been used as a wastewater disinfectant. However in contrast to chlorine, ozone reduces rather than increases effluent toxicity. Ozone also reduces effluent turbidity, colour, organic load and suspended solids content far more effectively than chlorine. These advantages combined with improved design of ozone generators have made ozone disinfection more viable.

This report contains the results of a study conducted to evaluate the factors affecting the efficiency of a complete mix reactor for use in ozone disinfection of primary effluents. The four stages of the study involved: reactor

modelling; establishment of a pilot plant; the conduction of ozone disinfection tests; and a statistical analysis of data collected.

II LITERATURE REVIEW

A. Historical Development

Ozone is a triatomic molecule of oxygen with the chemical formula O_3. It was first reported by M. van Mauren in 1785 and named by the German-Swiss chemist Christian Schonbein in 1840. The first method of artificially producing ozone was introduced in 1857 by the German engineer Werner von Siemens who manufactured the gas by electrical discharge. In the years following von Siemens discovery, several experimental plants utilizing ozone were established in Europe to treat drinking water supplies. However ozone was not employed at a permanent installation until 1893, when it was used to disinfect Rhine River water at Oudshorrn, Holland. Large scale water treatment plants utilizing ozone were then established at Weisbaden, Germany in 1901 and Nice, France in 1906. Since these early efforts the development of ozone disinfection has been continuous and there are now more than 1000 installations in 20 different countries utilizing ozone as a method for treating municipal water supplies.

Ozonation was first used as a method of wastewater treatment in 1930 to reduce BOD in municipal wastewater (McCarthy and Smith, 1974). Following this, no significant studies were conducted with respect to ozone treatment of wastewaters until 1967 (McCarthy and Smith, 1974). In 1967 ozone was used to treat wastewater on a pilot plant scale at the Eastern Sewage Works in the Redbridge Borough, London, England. Since then other significant pilot plant studies have been conducted in Chicago, Ill., which ozonated microstrained activated sludge effluent in 1970; Blue Plains, Washington, D.C., which ozonated various secondary effluents in 1970; Ft. Southworth, Louisville, Ky., which ozonated activated sludge effluents in 1971; Westerly wastewater treatment plant,

Cleveland, Ohio, which ozonated effluent for the purpose of disinfection in 1974; and Whitehorse, Yukon Territory where ozone was used to disinfect cold dilute wastewaters in 1978. At the present time the United States, which has 32 sewage treatment plants incorporating ozonation, is the only country which uses ozone as a method of wastewater treatment on a significant scale.

B. Effectiveness of Ozone Vs. Chlorine as a Disinfectant

Ozone is generally considered to be a better viral and bacterial disinfectant than chlorine (Pavoni et al., 1972; Nebel et al., 1973b; Rosen, 1976; Katzenelson and Biedermann, 1976; Lue-Hing et al., 1976; Farooq et al., 1977a). Tonelli et al., (1981) report that ozone is more effective than chlorine in inactivating aerobic spore formers, bulk T bacteriophage, fecal coliforms, fecal streptococci, Pseudomonas aeruginosa and salmonella. According to Farooq et al. (1978) a three to four decade reduction in E. coli counts may be obtained with a contact time of 2-5 minutes in the presence of an ozone residual of 0.25-0.30 mg/L. In contrast, Clarke et al. (1964) state that similar concentrations of chlorine residuals as monochloramine require between 150-200 minutes to achieve a two decade reduction in E. coli counts. Monochloramine is the predominant form of chlorine present in chlorinated wastewater effluents (White, 1978). Scaccia and Rosen (1978) discovered that an average ozone residual of 0.03 mg/L for a one minute contact period will provide an equivalent level of disinfection of enteric bacteria as a monochloramine residual of 4.5 mg/L for a 30 minute contact period.

According to Morris (1975), microorganism survival may be represented by:

$$N_t = N_o \exp(-\Lambda D) \qquad (1)$$

N_t = microorganism count at time t

N_o = microorganism count at time zero

D = disinfectant dose = Ct, (mg/L) (min.)

C = disinfectant concentration, mg/L

t = contact time, min.

Λ = specific lethality coefficient, $(mg/L)^{-1}$ $(min.)^{-1}$

The particular value $D = 1/\Lambda$ is called the lethal unit or lethe of the disinfectant under consideration. Lethe is the constant concentration of disinfectant needed to reduce N_o to N_o/e within a unit time. Hence, Λ may be defined as the number of lethes provided by a constant concentration of disinfectant in a unit time. The greater the value of Λ, the more effective a compound is as a disinfectant. Values of Λ as reported by Morris (1975) are listed in Table 1. These values indicate that ozone is a more effective disinfectant than chlorine in its various aqueous forms.

Ozone's superiority over chlorine is due to its high oxidation potential. Ozone has the highest oxidation potential of all commonly available chemical oxidants (Table 2). Since the disinfecting power of chemical compounds generally parallels their oxidation potential (Morris, 1975), ozone may be considered one of the most powerful chemical disinfectants currently available.

C. Toxicity of Chlorinated and Ozonated Effluents

Wastewater disinfection by the addition of chlorine has led to concerns involving the toxicity of chlorinated effluents and the formation of chlorinated organics. The toxicity of chlorinated effluents does not depend on the amount of chlorine added but rather on the chlorine residual remaining in the effluent (EPS, 1978). Chlorine residuals are known to persist for long periods of time in receiving waters and chlorinated effluents are known to be more toxic to aquatic organisms

TABLE 1 VALUES OF Λ AT 5°C, Λ IN $(mg/L)^{-1}\ (min)^{-1}$

Disinfectant	Enteric Bacteria	Amoebic Cysts	Viruses	Spores
O_3	500	0.5	5	2
HOCl as Cl_2	20	0.05	1.0 up	0.05
OCl^- as Cl_2	0.2	0.0005	< 0.02	<0.0005
NH_2Cl as Cl_2	0.1	0.02	0.005	0.001

TABLE 2 OXIDATION POTENTIAL OF CHEMICAL DISINFECTANTS

Disinfectant	Oxidation Potential (volts)
Ozone	2.07
Permanganate	1.67
Hypobromous acid	1.59
Chlorine dioxide	1.50
Hypochlorous acid	1.49
Hypoiodous acid	1.45
Chlorine gas	1.36
Bromine	1.09
Monochloramine	1.06
Iodine	0.54

than non-chlorinated effluents. Tables 3,4,5 and 6 summarize acute and chronic toxicity data of applied and residual chlorine to freshwater and marine organisms. From this data it is apparent that chlorine residuals as low or lower than 0.1 mg/L are toxic to aquatic organisms. Since many wastewater treatment plants maintain chlorine residuals for disinfection at 1.0 mg/L or greater (EPS, 1978; EHD, 1982), residual chlorine, after mixing may be present in many receiving streams in quantities highly toxic to aquatic life. In addition, chlorine residual concentrations non-toxic to fish and invertebrates have been found to be toxic to phytoplankton (EPS, 1978; Coneway and Styner, 1978; EHD, 1982). As a result, fish and invertebrate feeding movements may be indirectly affected by chlorine residuals.

Wastewater chlorination results in the formation of hundreds of chlorinated organics which may be carcinogenic and/or mutagenic (Rosen, 1976). Many of these compounds are not removed from drinking water supplies by traditional water treatment methods. Representative compounds formed during wastewater chlorination are listed in Table 7.

Numerous researchers have reported that ozonated effluents are non-toxic to aquatic organisms while at the same time reporting that chlorinated and non-disinfected effluents are (Ward and DeGraeve, 1978; Diaper, 1975; Arthur et al., 1975; Venosa and Ward, 1978; Hammond and Bishop, 1979; Tonelli et al., 1981). The non-toxic characteristic of ozonated effluents are due to ozone's extreme instability. This instability causes ozone to decompose rapidly in solution and hence prevents the maintenance of an ozone residual which may be toxic to aquatic organisms (Ward and DeGraeve, 1978; Farooq et al., 1977a; Tonelli et al., 1981).

Very little is known about the by-products formed during wastewater ozonation. Due to the non-toxic nature of ozonated effluents, it is commonly

TABLE 3 SELECTED FIELD STUDIES DEMONSTRATING EFFECTS OF CHLORINATED SEWAGE ON AQUATIC LIFE (EHD, 1982)

Location	Species	Results
Virginia James River	Numerous fish species including salmonids	Reduced diversity. Absence of Brook and Brown Trout at 0.02 mg/L chlorine
Michigan	Rainbow Trout	Toxic conditions at average chlorine of 0.014 mg/L
California Sacramento River	Chinook Salmon	Dechlorination required to halt fish kills
British Columbia	Sockeye Salmon Pink Salmon Coho Salmon Rainbow Trout	Mortalities common when chlorine residuals in excess of 0.02 mg/L
Ontario	Rainbow Trout	96-hr LC_{50} ranged from 0.01 to 0.09 mg/L residual chlorine in secondary effluent

TABLE 4 ACUTE TOXICITY OF RESIDUAL CHLORINE TO FRESHWATER FISH (EPS, 1978)

Species	Concentration (mg/L) - Effect
Salmo gairdneri	0.014-0.029, 96 hr. LC_{50}
Salvelinus fontinalis	0.146, 10°C, 96 hr. LC_{50}
Salvelinus fontinalis	0.102, 20°C, 96 hr. LC_{50}
Salvelinus fontinalis	0.083, 7 day LC_{50}
Pimephales promelas	0.086-0.13, 96 hr. LC_{50}
Pimephales promelas	0.082-0.115, 7 day LC_{50}

TABLE 5 CHRONIC TOXICITY OF RESIDUAL CHLORINE TO FRESHWATER ORGANISMS (EPS, 1978)

Organism	Concentration (mg/L) - Effect
Oncorhynchus gorbuscha	0.05, maximum non-lethal
Oncorhynchus kisutch	0.05, maximum non-lethal
Salvelinus fontinalis	0.015, absent in streams
Micropterus dolomieui	0.10, absent in streams
Pimephales promelas	0.0165, safe concentration
Salmo gairdneri	0.001, slight avoidance

TABLE 6 TOXIC EFFECTS OF CHLORINATED EFFLUENTS ON MARINE INVERTEBRATES (EHD, 1982) (cont'd)

Species	Chlorine Dosage (mg/L)	Test Duration	Effect
Strongylocentrotus purpuratus (gametes)	0.11	5 min.	100% inhibition of fertilization
Urechis caupo (gametes)	0.2	5 min.	22% inhibition of fertilization
	1.0	5 min.	100% inhibition of fertilization
Phragmatopoma californica (sperm)	0.2	5 min.	22% loss of motility
	1.0	5 min.	86% loss of motility
Eliminus modestus	2.0	10 min.	death and inhibited growth
Balanus sp.	2.5	5 min.	80% mortality
Acartia tonsi	2.5	5 min.	90% mortality
Melita nitida	2.5	5 min.	near 100% mortality 96 hr. after exposure
Palaemonetes pugio	2.5	5 min.	near 100% mortality 96 hr. after exposure
Anemones	10	1,2,4,8 hr/ day for 10 days	none
	2.5	5 days	100% mortality
	1.0	15 days	100% mortality

TABLE 6 TOXIC EFFECTS OF CHLORINATED EFFLUENTS ON MARINE INVERTEBRATES (EHD,1982)

Species	Chlorine Dosage (mg/L)	Test Duration	Effect
Barnacles	10	1,2,4,8 hr/ for 10 days	95-100% mortality
	2.5	4 days	100% mortality
	1.0	7	100% mortality
Homarus americanus	2.02	48-h	50% mortality
Crassostrea virginica (larvae)	0.005	48-hr	50% mortality
Crassostrea virginica (juvenile)	0.023	96-hr	50% mortality

TABLE 7 CHLORINATED ORGANICS IN WASTEWATER EFFLUENTS (EPS, 1978) (cont'd)

chloroform	trichlorocumene (222)
dibromochloromethane	tetrachloroethylstyrene (268)
dichlorobutane	trichlorodimethoxybenzene (240)
3-chloro-2-methylbut-1-ene	tetrachloromethoxytoluene (258)
chlorocyclohexane (118)	dichloroaniline derivative (205)
chloroalkyl acetate	dichloroaromatic derivative (249)
o-dichlorobenzene	dichloroacetate derivative (203)
tetrachloroacetone	trichlorophthalate derivative (296)
p-dichlorobenzene	tetrachlorophthalate derivative (340)
chloroethylbenzene	5-chlorouracil
pentachloroacetone	5-chlorouridine
hexachloroacetone	8-chlorocaffeine
trichlorobenzene	6-chloroguanine
dichloroethyl benzene	8-chloroxanthine
chlorocumene (154)	2-chlorobenzoic acid
n-methyl-trichloroaniline (209)	5-chlorosalicylic acid
dichlorotoluene	4-chloromandelic acid
trichlorophenol	2-chlorophenol
chloro-a-methyl benzyl alcohol	4-chlorophenylacetic acid
dichloromethoxytoluene	4-chlorobenzoic acid
trichloromethylstyrene (220)	4-chlorophenol
trichloroethyl benzene (208)	3-chlorobenzoic acid
dichloro-bis(ethoxy)benzene (220)	3-chlorophenol

TABLE 7 CHLORINATED ORGANICS IN WASTEWATER EFFLUENTS (EPS, 1978)

dichloro-a-methyl benzyl alcohol (190)	4-chloroesorcinol
trichloro-n-methylanisole	3-chloro-4-hydroxy-benzoic acid
tetrachlorophenol	
trichloro-a-methyl benzyl alcohol	

thought that toxic by-products formed during wastewater ozonation rapidly decompose to carbon dioxide and water or other non-toxic compounds. Formaldehyde, methylglyoxal, and glyoxylic, pyruvic, oxalic, acetic and formic acids are considered to be major by-products found in ozonated wastewaters prior to their decomposition to carbon dioxide and water (Kuo et al., 1977; Yamada and Somiya, 1980).

D. Contact Time Requirements for Ozone and Chlorine Disinfection

Chlorination inactivates microorganisms relatively slowly with respect to contact time. The minimum contact time required in wastewater chlorination is approximately 15 minutes (Metcalf and Eddy, 1979). In contrast, ozonation produces a very rapid inactivation of microorganisms within a short period of time and then levels off (Katzenelson et al., 1974). This occurs because ozone is immediately reactive in solution whereas chlorine must undergo hydrolysis first (Farooq et al., 1978). Hence contact time is of secondary importance to mass transfer in ozone disinfection (McCarthy and Smith, 1974). Consequently ozone inactivates microorganisms rapidly in the presence of an ozone residual. Contact times sufficient to establish a residual (1-10 min.) are found to be adequate for disinfection purposes (Rosen, 1976; Farooq et al., 1978).

E. Consideration of Disinfection Alternatives

Chlorination-dechlorination practices are known to produce effluents which are less toxic than unchlorinated effluents (Roberts and Vajdic, 1974). Although dechlorination reduces chlorine induced fish toxicity, its ability to eliminate chlorinated organics still requires investigation (EPS, 1978). Recent studies indicate that when dechlorination is practiced, ozonation becomes economically competitive with chlorination (Budde et al., 1977; EPS, 1978).

The bactericidal efficiency of chlorine dioxide is comparable to that of chlorine at neutral pH values and increases with pH, thus making it a more effective wastewater disinfectant than chlorine. However chlorine dioxide is a less effective disinfectant than ozone, probably due to its lower oxidation potential (1.50 vs. 2.07 volts). Chlorine dioxide does not react with nitrogenous compounds to form toxic chloramines but it does reduce to the chlorite ion which is suspected to be toxic to humans (Roberts and Vajdic, 1974).

Bromine chloride is considered to be a possible wastewater disinfectant. Its disinfection properties are considered to be superior to those of chlorine while at the same time it produces effluents which are less toxic to aquatic life (EPS, 1978). However the application of bromine chloride as a wastewater disinfectant is still in the development stage.

Iodine and bromine are not commonly used as disinfectants. If used as a wastewater disinfectant bromine would require massive dosages in comparison to chlorine or ozone, thereby making its use uneconomical (White, 1978). Iodine is considered to be too expensive for use on a large scale. It has been suggested for use in isolated areas because of its ease of transportation (EPS, 1978). However its high cost combined with ozone's ability to be generated on site make it a less practical disinfectant than ozone in large scale application.

Ultraviolet (UV) radiation is a physical rather than chemical method of disinfection. UV radiation is similar to ozonation in that it produces effluents which are less toxic to aquatic life than chlorinated effluents (EPS, 1978). UV light is effective only when the growth of slime layers on the UV lamp is controlled. As a result, it is not a practical method of wastewater disinfection.

F. Effect of Wastewater Quality on Ozone Disinfection

Ozone is an extremely powerful, non-selective oxidizing agent which will react with many organic and inorganic compounds found in wastewater (Rice et al., 1981). The presence of such material creates an extraneous ozone demand which reduces the disinfection efficiency of ozone. Consequently as effluent quality decreases higher ozone dosages and longer contact times are needed to maintain a given level of disinfection (Nebel et al., 1976a; Rosen, 1979; Tonelli et al., 1981).

Suspended solids concentration is considered to be a major parameter affecting disinfection with ozone. This is due to the ability of suspended solids to shield microorganisms and thereby prevent ozone from coming in direct contact with them. Consequently ozone concentrations which would normally provide adequate disinfection achieve only partial microorganism inactivation in the presence of suspended solids (Ghan et al., 1976; Nebel et al., 1976b; Howser et al., 1978; Sproul et al., 1979).

The effect of wastewater temperature on ozone disinfection has not been fully established. Studies conducted by Farooq et al. (1977c) and Engelbrecht, et al. (1979) report that microorganism inactivation increases with increasing temperature. They explained their findings by stating that even though elevated temperatures increase the rate of ozone decomposition, it also increases ozone's chemical reaction rates. As a result the rate of microorganism inactivation increases. In contrast, Given and Smith (1982) discovered that microorganism inactivation increases as temperature decreases, which may be attributed to a lower reaction rate with the ozone consuming materials. This may also be due to a reduction in the rate of ozone residual decomposition as temperature decreases.

Furthermore, ozone maintains its chemical activity to -70°C (Kinman, 1974; Katzenelson et al., 1974).

Organic matter impedes ozone's ability to disinfect wastewaters (Ghan et al., 1976; Guirguis et al., 1976; Katzenelson and Biedermann, 1976; Bollyky and Siegel, 1977). This is due to the ability of organic matter to consume ozone in rapid oxidation reactions, thereby reducing the amount of ozone available for microorganism inactivation. Hence as wastewater BOD and COD increases, the ozone dosages required to achieve disinfection increases.

Ozone decomposition is considerably accelerated by increased alkalinity (Hoigne and Bader, 1975). Under alkaline conditions, decomposition products of ozone such as the hydroxl radical ($OH^{\bullet}$) become the important oxidant. Carbonate and bicarbonate ions react with the hydroxl radical in electron transfer reactions (equations 2 and 3) and prevent it from being an effective disinfectant (Hoigne and Bader, 1975).

$$OH^{\bullet} + CO_3^{-2} \rightarrow OH^{-} + CO_3^{\bullet -} \quad (2)$$

$$OH^{\bullet} + HCO_3^{-} \rightarrow OH^{-} + HCO_3^{\bullet} \quad (3)$$

The hydroxyl radical reacts with the carbonate ion much more quickly than with the bicarbonate ion. As a result, the protecting effect of a given amount of total carbonate on microorganism survival should increase in the pH range above 9 where the equilibrium shifts from the bicarbonate ion toward the carbonate ion (Hoigne and Bader, 1975).

Microorganism inactivation appears to be more directly related to the ozone residual than wastewater pH. However, pH influences the rate of ozone decomposition which in turn affects the ozone residual (Farooq et al., 1977b). The ozone decomposition rate is much more rapid in aqueous solutions at high pH

values. This is due to the catalytic activity of the hydroxl ion (Farooq et al., 1977a). Gurol and Singer (1980) report that the rate of ozone decomposition in aqueous solutions depends on the ozone and hydroxl ion concentrations (equation 4).

$$r_{O_3} = \frac{-d(O_3)}{dt} = k_o (OH^-)^{0.5} (O_3)^2 \quad (4)$$

A lower ozone residual results with increasing pH because of the increased rate of ozone decomposition which in turn decreases the degree of microorganism inactivation attainable.

The reaction between ozone and ammonia at pH values less than 9 is very slow (Hoigne and Bader, 1978). At pH values above 9 the rapid decomposition of ozone to radicals precedes the oxidation of ammonia. Of the radicals formed during ozone decomposition only the hydroxl radical ($OH^{\bullet}$) is relevant in the oxidation of ammonia. However ammonia consumes the hydroxl radical in a reaction with a low rate constant compared with those of carbonate ions and organics. This permits free ammonia to significantly inhibit other oxidation reactions only when present in concentrations at least comparable to those of the carbonate ions and the sum of organic solutes (Hoigne and Bader, 1975). Such concentrations are not found in domestic wastewaters and as a result ammonia does not inhibit ozone disinfection in most wastewaters.

G. Ozone Mass Transfer and Mixing Systems

Ozone residuals are considered to be one of the dominant factors controlling the inactivation of microorganisms during wastewater ozonation. However the presence of ozone bubbles with an ozone residual provides a higher degree of inactivation than a given ozone residual alone (Farooq et al., 1977a;

Engelbrecht et al., 1979). This occurs because microorganisms tend to congregate at the gas-liquid interface of ozone bubbles and are exposed to higher ozone concentrations than are present in the bulk solution (Farooq et al., 1978). As a result mixing systems should promote the transfer of ozone from the gas to the liquid phase and ensure a complete dispersal of ozone bubbles throughout the wastewater (Masschelein et al., 1975).

The mass transfer of ozone from the gas to the liquid phase may be explained by the film theory of gas transfer (Figure 1). In ozonation, molecules of ozone are transferred from the bulk gas phase to the outer gas-film interface by turbulent mixing and eddy diffusion. Molecular diffusion moves the molecules to the gas-liquid interface where they dissolve into the stagnant liquid film. The molecules diffuse through the stagnant liquid film to the film's outer boundary where they are transported to the bulk liquid phase by turbulent mixing.

The rate of ozone mass transfer by molecular diffusion is slow compared with turbulent mixing and eddy diffusion, permitting the stagnant films to control the rate of mass transfer (Canale and Weber, 1972). The rate of ozone mass transfer into the liquid phase may be represented by equation 5 (McCarthy and Smith, 1974).

$$\ln\left(\frac{C-C_t}{C-C_o}\right) = -K_L a \cdot t \qquad (5)$$

C = equilibrium concentration of the solute gas in the liquid phase corresponding to the pressure of the solute gas in the gas phase, (ML^{-3})

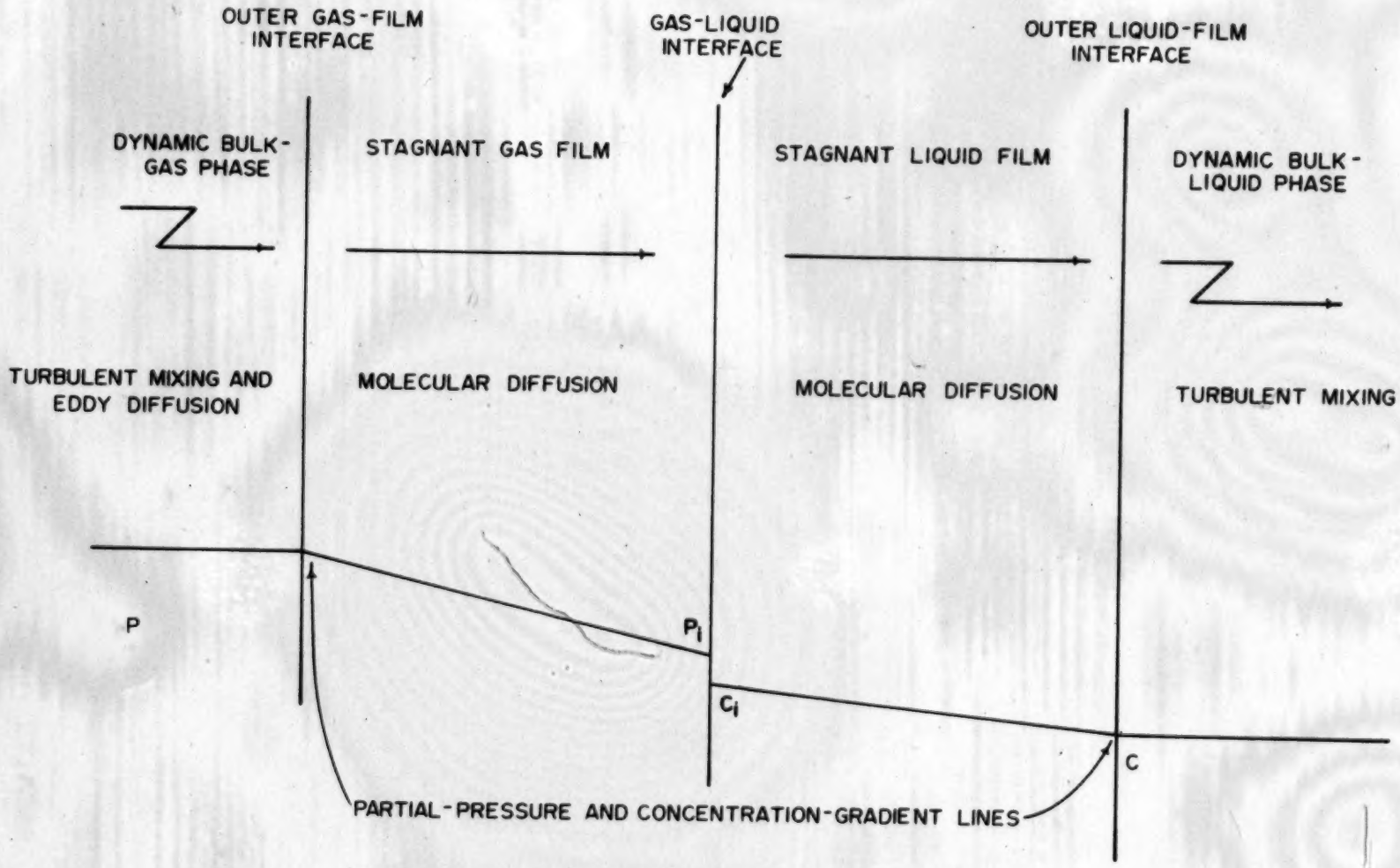

FIGURE 1: FILM THEORY OF GAS TRANSFER

C_o, C_t = concentration of the solute gas in the liquid phase at time zero and t respectively, (ML^{-3})

K_L = overall mass transfer coefficient, (LT^{-1})

$K_L a$ = overall volumetric mass transfer coefficient (T^{-1})

a = $\frac{A}{V}$, (L^{-1})

A = area across which diffusion is taking place, (L^2)

V = volume of system, (L^3)

t = time, (T)

From equation 5 it is apparent that ozone mass transfer may be increased by increasing the ozone concentration differences between the gas and liquid phases (McCarthy and Smith, 1974).

The main types of ozone mixing systems currently available include; injectors, porous diffusers and turbine mixers (Bowers et al., 1973; Bollyky, 1976). Injection systems consist of jet pumps operating on the venturi principle. The two basic types of injection systems are: total injection in which the whole flow of water under treatment, at low pressure, is mixed with ozonated gas; and partial injection where a portion of the water flow, at high pressure, is mixed with ozonated gas and then dispersed through the bulk water flow (Bowers et al., 1973).

Porous diffuser elements are generally porous ceramic pipes or plates, stainless steel floors or plastic dispensers. These diffusers are installed at the bottom of contact chambers and inject ozonated gas either countercurrently or concurrently to liquid flow. Countercurrent injection is generally considered to be more effective than concurrent injection (Masschelein et al., 1975).

Turbine mixers combine gas and liquid streams through the mechanical mixing process. High speed agitation has been found to increase the rate of ozone

decomposition thereby reducing the ozone residual and the degree of disinfection attained. However the use of turbine mixers within the range 0 to 345 rpm has little effect on the rate of ozone decomposition and ozone disinfection efficiency (Farooq et al., 1978).

Advantages and disadvantages of these systems are listed in Table 8.

H. Ozone Requirements for Wastewater Disinfection

Although the presence of an ozone residual is the controlling factor in wastewater disinfection, the amount of ozone required is generally reported as the applied ozone dosage (Farooq et al., 1978). Table 9 lists ozone dosages and contact times required to treat and disinfect various types of water and wastewater. These values indicate that ozone dosages required for disinfection and treatment vary with wastewater quality.

I. Wastewater Characteristics

The characteristics of raw and treated wastewaters vary considerably depending on the wastewater sources and methods of treatment. Table 10 lists ranges of values for physical, chemical and bacteriological characteristics of raw municipal wastewaters and primary effluent. The bacteriological characteristics of primary effluents are assumed to be approximately 50% of those of raw wastewater (Pelczar and Reid, 1972).

TABLE 8 ADVANTAGES AND DISADVANTAGES OF OZONE MIXING SYSTEMS (Masschelein et al., 1975, 1976; Bollyky, 1976) (cont'd)

Mixing System	Advantages	Disadvantages
Injection	Agitation is attained without the use of mechanical equipment	Tendency for vertical travel of bubbles with only partial direct ozone bubble to liquid contact
		High power requirements
		Hydraulic head losses occur
		Limited system control is available due to the importance of maintaining a required gas to liquid ratio
		Partial injection permits only part of the wastewater to be treated by direct exposure to ozonated gas
Porous Diffusers	Static operation without mechanical or electrical maintenance	Tendency for vertical channeling of bubbles without intimate mixing between ozone bubbles and wastewater
	No hydraulic head loss in liquid under treatment	Clogging of pores causes uneven distribution of ozonated gas
	Flexible system controls are available	
	Gravity flow conditions may be employed.	

TABLE 8 ADVANTAGES AND DISADVANTAGES OF OZONE MIXING SYSTEMS (Masschelein et al., 1975, 1976; Bollyky, 1976)

Mixing System	Advantages	Disadvantages
Turbine Mixers	Gravity flow conditions may be employed	High mixing speeds increase the rate of ozone decomposition
	No hydraulic head loss in liquid under treatment	High power requirement
	Flexible system controls are available	
	Excellent mixing	
	Mass transfer coefficients for ozone increases with power input per unit volume.	

TABLE 9 COMPARISON OF OZONE DOSES REPORTED ON WATER AND WASTEWATER TREATMENT (McCarthy and Smith, 1974)

Dose, in milligrams	Contact time, in minutes	Type of Water-wastewater treated	Objective or result
0.5-1	5-10	Good quality ground water	Disinfection
2-3	5-10	Good quality surface water	Disinfection
3-4	5-10	Poor quality surface water with filtration	Disinfection
1.5-2	not given	Treated water	Disinfection
2-5	5	Treated water	Disinfection
10-100	not given	Primary wastewater and storm water overflow	Treatment
>50	not given	Secondary wastewater effluent	Potable quality
15	22	Secondary wastewater effluent	Disinfection 200 fecal coliforms/100 ml
15	5	Secondary wastewater effluent	f-2 virus destruction
5	1.6	Tertiary wastewater effluent	Complete bacterial disinfection
5	<	Secondary wastewater effluent	Disinfection <1,000 coliforms/100 ml
550	90	Inoculated raw wastewater	Sterility
92	30	Inoculated autoclaved wastewater	Sterility

TABLE 10 WASTEWATER CHARACTERISTICS (CRC Handbook of Environmental Control, 1974; Metcalf and Eddy, 1979).

Characteristic	Raw Wastewater	Primary Effluent
5-day BOD (mg/L)	100-470	70-390
Suspended solids (mg/L)	100-500	40-130
pH	6.5-9.4	-
Alkalinity (mg/L $CaCO_3$)	50-200	-
Total Coliforms (count/100 ml)	$2.0 \times 10^6 - 3.3 \times 10^7$	$1.0 \times 10^6 \times 1.7 \times 10^7$
Fecal Coliforms (count/100 ml)	$3.4 \times 10^5 - 1.7 \times 10^7$	$1.7 \times 10^5 - 8.5 \times 10^6$

III PROJECT OBJECTIVES

There is little information available on the factors affecting ozone disinfection of wastewaters using complete mix reactors. The overall objective of this project was to provide basic information about the performance of complete mix reactors used for wastewater disinfection. To achieve this a number of sub-objectives were established. These were:

1. to conduct dye studies on a turbine reactor to determine conditions which produced complete mixing;
2. to develop a data base for ozone disinfection tests using a complete mix reactor;
3. to conduct a statistical analysis on the data to establish predictive empirical relationships for bacterial survival; and
4. to compare these relationships for complete mix reactors to relationships developed for porous diffuser reactors by Given and Smith (1982).

IV PROJECT COMPONENTS

A. Reactor Modelling

Figure 2 presents a sketch of the reactor used in this project. The reactor was designed to provide rapid intense complete mixing of ozonated gas and primary effluent. Complete mix reactors exhibit an exponential decay in dye concentrations when subjected to a slug dye input (Figure 3). This decay may be modelled by equation 6 (Metcalf and Eddy, 1979).

$$C_T = C_o e^{-t/\theta} \quad (6)$$

C_T = theoretical dye concentration at time t

C_o = initial dye concentration = M/V

M = mass of dye injected

V = reactor volume

t = time

θ = nominal detention time = V/Q

Q = liquid flowrate

The reactor had a maximum turbine speed of 360 rpm. Slug dye studies utilizing methylene blue dye were conducted at speeds of 60,180 and 300 rpm at various depths and retention times to determine what operating conditions were required to achieve complete mixing. Outflow dye concentrations were monitored with time using a Bausch and Lomb Spectronic 20 spectrophotometer.

The Spectronic 20 measures the amount of light transmitted through a non-turbid coloured solution. The concentration of the colour causing substance may be determined by preparing a series of standard concentrations and measuring their percent transmittance (% T). A plot of log (%T) versus concentration may

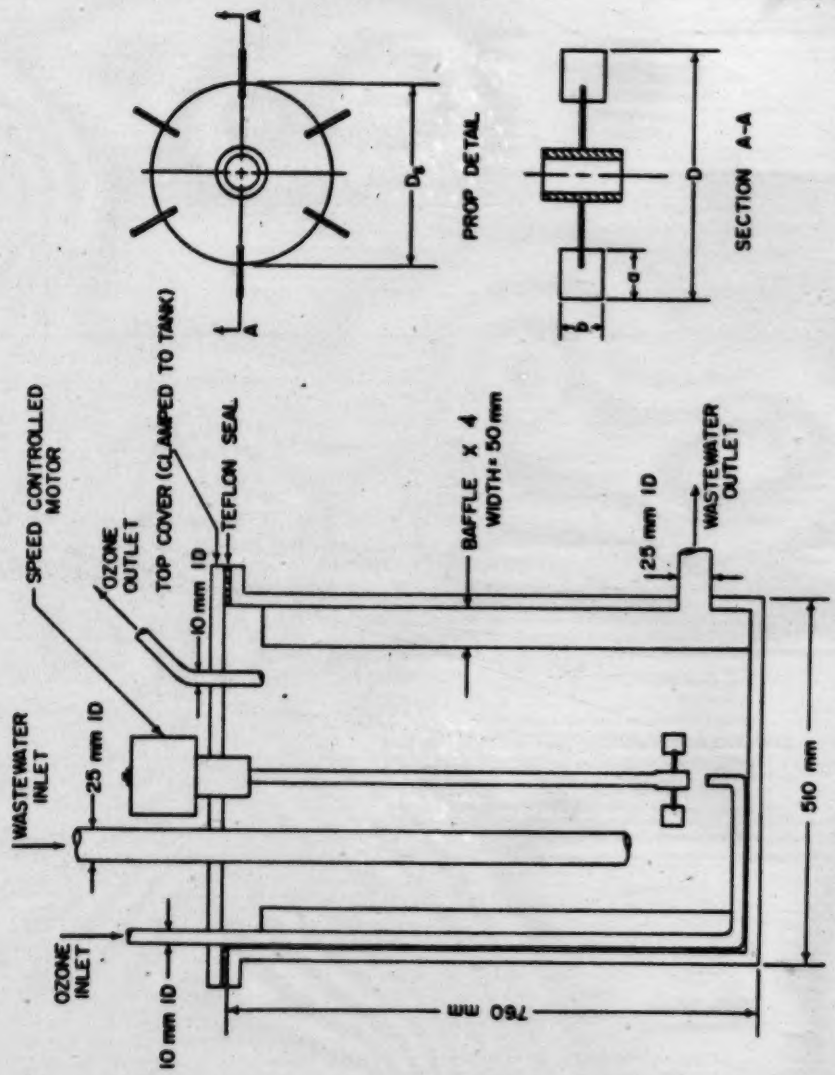

FIGURE 2: COMPLETE MIX REACTOR

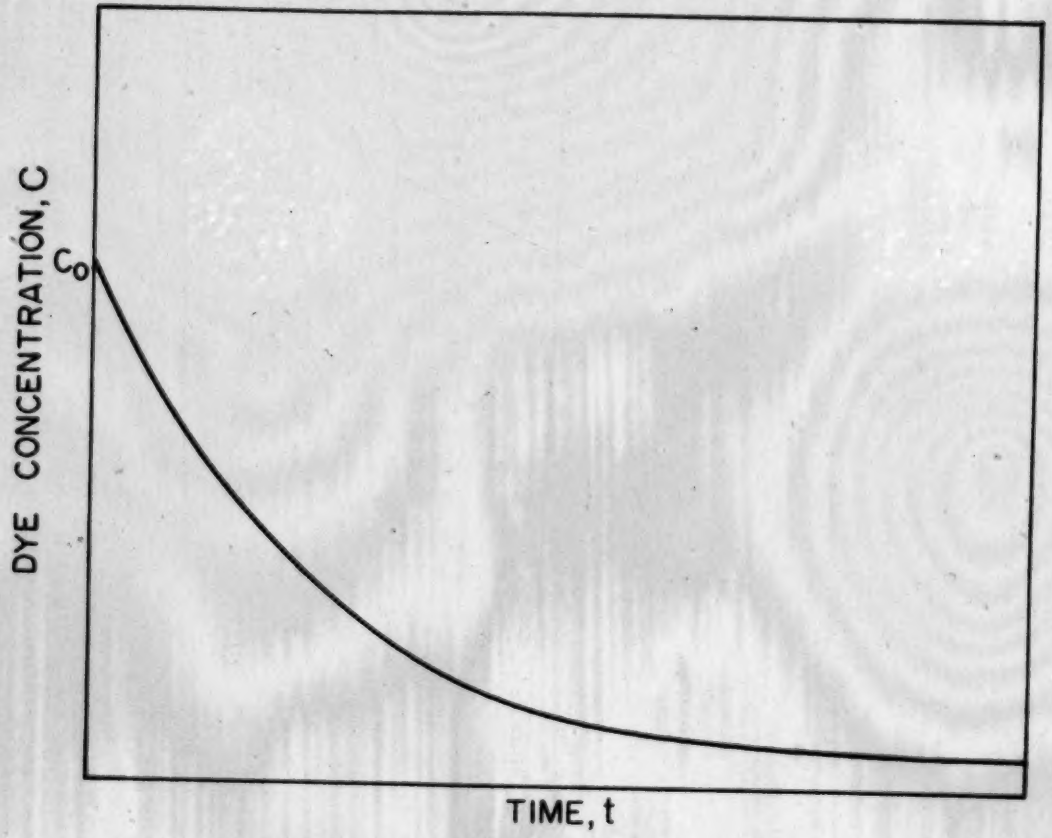

FIGURE 3: EXPONENTIAL DECAY CURVE FOR COMPLETE MIX REACTORS SUBJECT TO SLUG DYE INPUTS

then be produced to provide a calibration curve. This permits the outflow dye concentration to be determined by measuring the percent transmittance (%T) of the reactor outflow at various times and determining the concentrations from the calibration curve.

Figure 4 presents a schematic diagram of the dye testing system and Plate 1 illustrates the actual system. Clear sump water was used as a test liquid. The test liquid was pumped into the reactor using a one-half (0.5) horsepower sump pump. The rate of liquid flow was measured using a Signet Scientific magnetic flowmeter and was controlled using a fine adjustment valve. The liquid depth within the reactor was also controlled using a fine adjustment valve at the reactor outlet. Slug inputs of methylene blue dye were injected into the reactor using a rapid discharge syringe. Dye samples were collected in 100 ml Nalgene plastic sample bottles.

B. Laboratory Facilities and Procedures

Experimental facilities were established at the City of Edmonton's Goldbar Wastewater Treatment Plant. The facilities consisted of two trailers; an experimental trailer and a microbiology trailer (Plate 2, 3, 4). The entire ozone plant was contained within the experimental trailer. Primary effluent was pumped from treatment facilities into a holding tank within the experimental trailer. Figure 5 presents a schematic diagram of the ozone pilot plant.

Ozone was produced by passing oxygen gas through a Union Carbide Linde ozone generator (Model no. SG-4060). A Precision Scientific 63111 wet test meter was used to measure the volume of ozonated carrier gas injected into the complete mix reactor during each test. The primary effluent flowrate was measured using a Signet Scientific magnetic flowmeter and controlled by a Sterling variable speed pump.

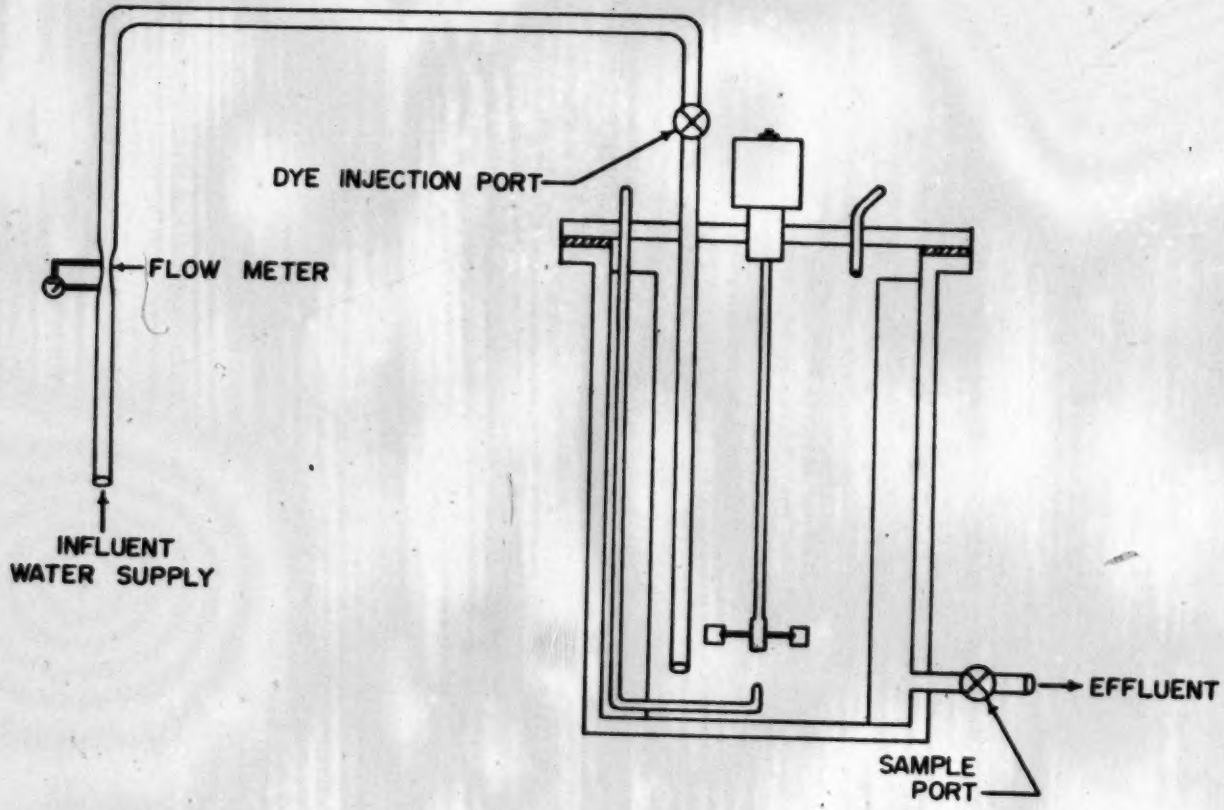

FIGURE 4: SCHEMATIC DIAGRAM OF THE DYE TESTING SYSTEM

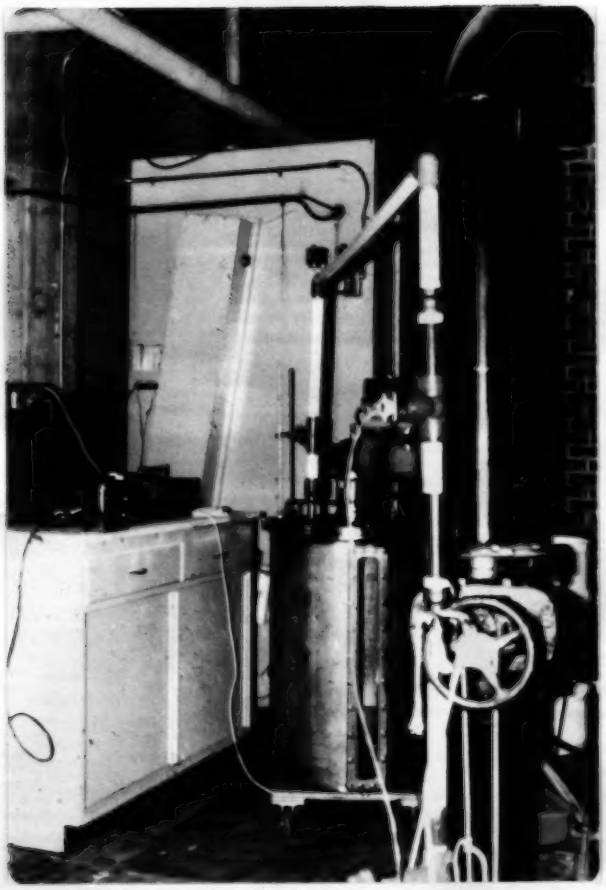

PLATE 1: DYE TESTING SYSTEM

PLATE 2: EXPERIMENTAL TRAILER

PLATE 3: EXPERIMENTAL TRAILER

PLATE 4: MICROBIOLOGY TRAILER

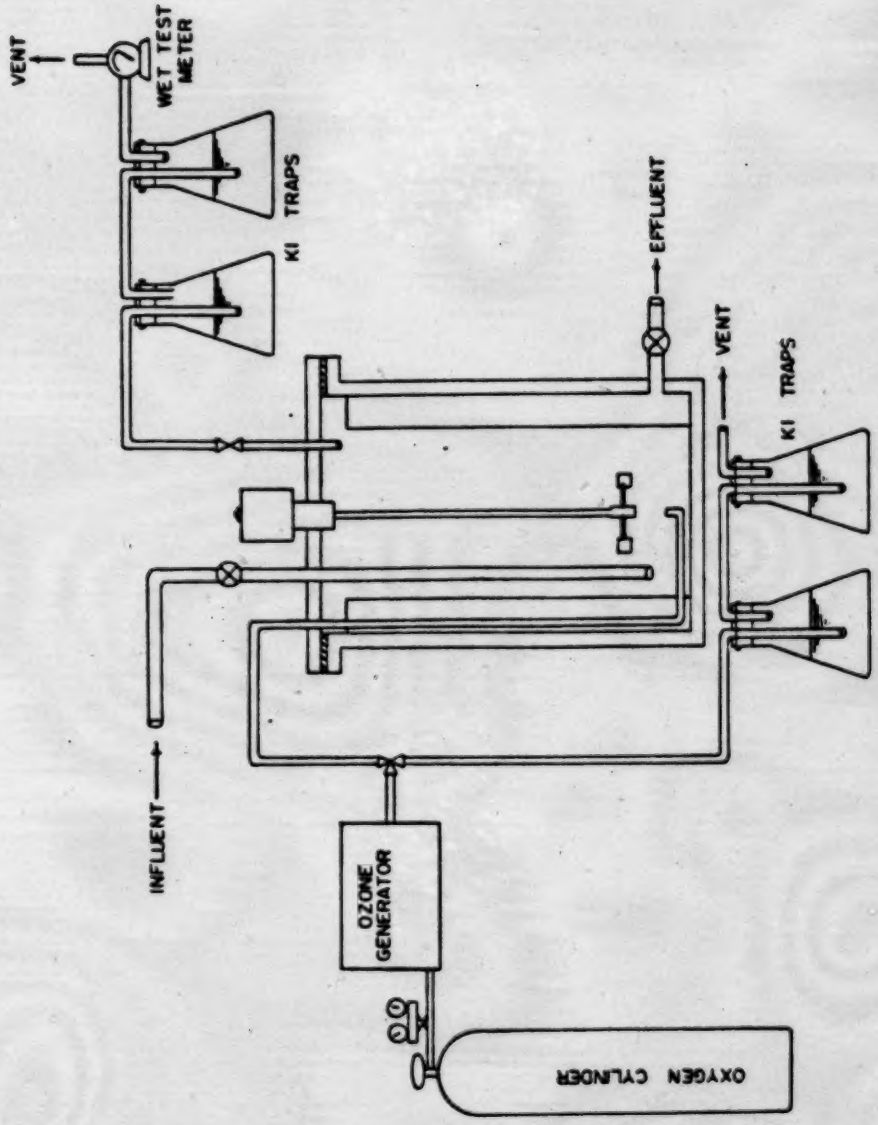

FIGURE 5: SCHEMATIC OF OZONE PILOT PLANT

It was necessary to calibrate the ozone generator daily to determine the concentration of ozone in the influent carrier gas. Following calibration, primary effluent was pumped into the reactor and valves adjusted to establish the operating depth and liquid flowrate. The ozone generator was then permitted to warm-up for a period of five minutes according to its operating instructions. While the generator was warming up two primary effluent samples were collected from the holding tank. One sample was used to determine the physical/chemical characteristics of the primary effluent and the other as a non-disinfected bacteriological sample. Immediately following the collection of these samples, wastewater and atmosphere temperature, barometric pressure and liquid temperature in the wet test meter were measured and recorded.

Ozonation was then started with two more non-disinfected bacteriological samples being collected from the holding tank at equally spaced intervals during the test. The three non-disinfected samples were used to establish an average estimate of bacterial quality of the primary effluent prior to disinfection by establishing initial average total and fecal coliform counts per 100 ml. Following each retention period two samples were collected from the reactor. One sample was used to determine bacterial quality after disinfection while the other was used to measure the ozone residual in the disinfected effluent. The duration of each test was selected to ensure that the original volume of primary effluent present in the reactor at the start of each test had discharged almost completely from the reactor by the end of each test. The theory of complete mix reactors (equation 6) predicts that less than 0.5% of the original material in a complete mix reactor will be present after six complete retention periods. Based on this theory a test duration of six complete retention periods was selected. During each test the manometric pressure reading on the wet test meter was recorded. At the end of

each test gas volume was recorded to determine the flowrate of ozonated carrier gas. Prior to the start of the next test the holding tank was drained and refilled and the reactor was bled of all ozonated effluent and gas.

The ozonated gas used in this study consisted mainly of the oxygen carrier gas with only a small portion being ozone. The effects of the oxygen carrier gas on bacterial survival were examined through the use of control tests. The control tests used the same procedures as the disinfection tests but employed pure oxygen gas rather than ozonated gas.

C. Wastewater Analysis

Non-ozonated samples collected from the holding tank were analyzed to determine the physical, chemical and bacteriological characteristics of the primary effluent prior to disinfection. Included in these analyses were measurements of temperature, turbidity, suspended solids, pH, alkalinity, 5-day BOD and total and fecal coliform counts. Ozonated samples were analyzed for total and fecal coliform counts.

Turbidity and pH measurements were conducted using a Hach 2100A turbidimeter and a Hach pH meter. Suspended solids, alkalinity, 5-day BOD, total and fecal coliform counts were determined according to Standard Methods procedures 208D, 403, 507, 909A and 909C (Am. Public Health Assoc., 1976). All bacterial samples were analyzed immediately and all physical/chemical samples were analyzed as soon as possible after collection.

D. Applied Ozone Dose, Utilized Ozone Dose and Ozone Residual

The applied and utilized ozone dose were calculated using equations 7 and 8.

$$D = C_1 \times Q_G/Q_L \qquad (7)$$

$$U = (C_1 - C_2) \times Q_G/Q_L \quad (8)$$

D = applied ozone dose, mg/L

U = utilized (absorbed) ozone dose, mg/L

C_1 = concentration of ozone in influent carrier gas, mg/L

C_2 = concentration of ozone in effluent carrier gas, mg/L

Q_G = flowrate of ozone carrier gas, L/min

Q_L = primary effluent flowrate, L/min

C_1 and C_2 were determined by discharging the influent and effluent carrier gas through potassium iodide (KI) solutions. Ozone reacts with potassium iodide to liberate iodine. The number of milli-equivalents of iodine was determined by titrating with a phenylarsene oxide solution using a Penwalt Wallace and Tierman amperometric titrator (series A-790). Knowing the number of milli-equivalents of iodine released and the carrier gas flowrate it was possible to determine C_1 and C_2.

The ozone residual in the disinfected primary effluent was measured by collecting an ozonated sample and immediately adding potassium iodide. The iodine released was then titrated with a phenylarsene oxide solution using an amperometric titrator. The ozone residual was calculated using equation 9.

$$R = \frac{N \times V_1 \times EW}{V_2} \quad (9)$$

R = ozone residual in disinfected effluent, mg/L

N = titrant normality

V_1 = volume of titrant used, ml

EW = equivalent weight of ozone, mg/meq

V_2 = volume of disinfected primary effluent titrated, L.

E. Data Analysis

The MIDAS (Michigan Interactive Data Analysis System) computerized packaged program was used to statistically analyze the data to determine (Fox and Guire, 1976; Statistical Research Laboratory, 1976):

1. the relationships between the dependent variables; total and fecal coliform survival ratios and the independent variables; applied ozone dose, ozone utilized, ozone residual, turbidity, temperature, BOD, suspended solids, alkalinity, pH and initial coliform numbers;
2. the effect of different operating conditions on disinfection efficiency; and
3. the effect of primary effluent characteristics, applied ozone dose and ozone utilized on the establishment of an ozone residual and the quantity of ozone utilized.

MIDAS was chosen as the method of data analysis because of its ability to manipulate large volumes of repetitive data containing many different variables.

V. RESULTS

A. Dye Tests

Figures 6 through 29 compare the measured dye decay curves with theoretical curves for various turbine speeds and operating conditions. Appendix 1 contains the data points for each curve. These figures show that not all the methylene blue dye injected into the reactor during each test was completely recovered.

Figures 6 through 29 indicate that the reactor approaches complete mix conditions most closely at turbine speeds of 300 rpm for depths of 375 and 500 mm. As a result all tests in this project were conducted at these operating conditions to ensure complete mixing and at the same time remain under the maximum turbine speed of 345 rpm recommended by Farooq et al., (1978).

B. Control Tests

Tables 11 and 12 contain the results of control tests conducted to examine the effect of oxygen on total and fecal coliform survival ratios. The results indicate that the application of oxygen does reduce the total and fecal coliform survival ratios.

C. Disinfection Results

The disinfection results for total coliforms (TC) and fecal coliforms (FC) are illustrated in Figures 30 to 35. These figures present log-log plots of total and fecal coliform survival ratios (N/N_o) versus applied ozone dose (D), ozone utilized (U) and ozone residual (R). The data points for these curves are listed in Appendix 2 along with primary effluent characteristics and operating data.

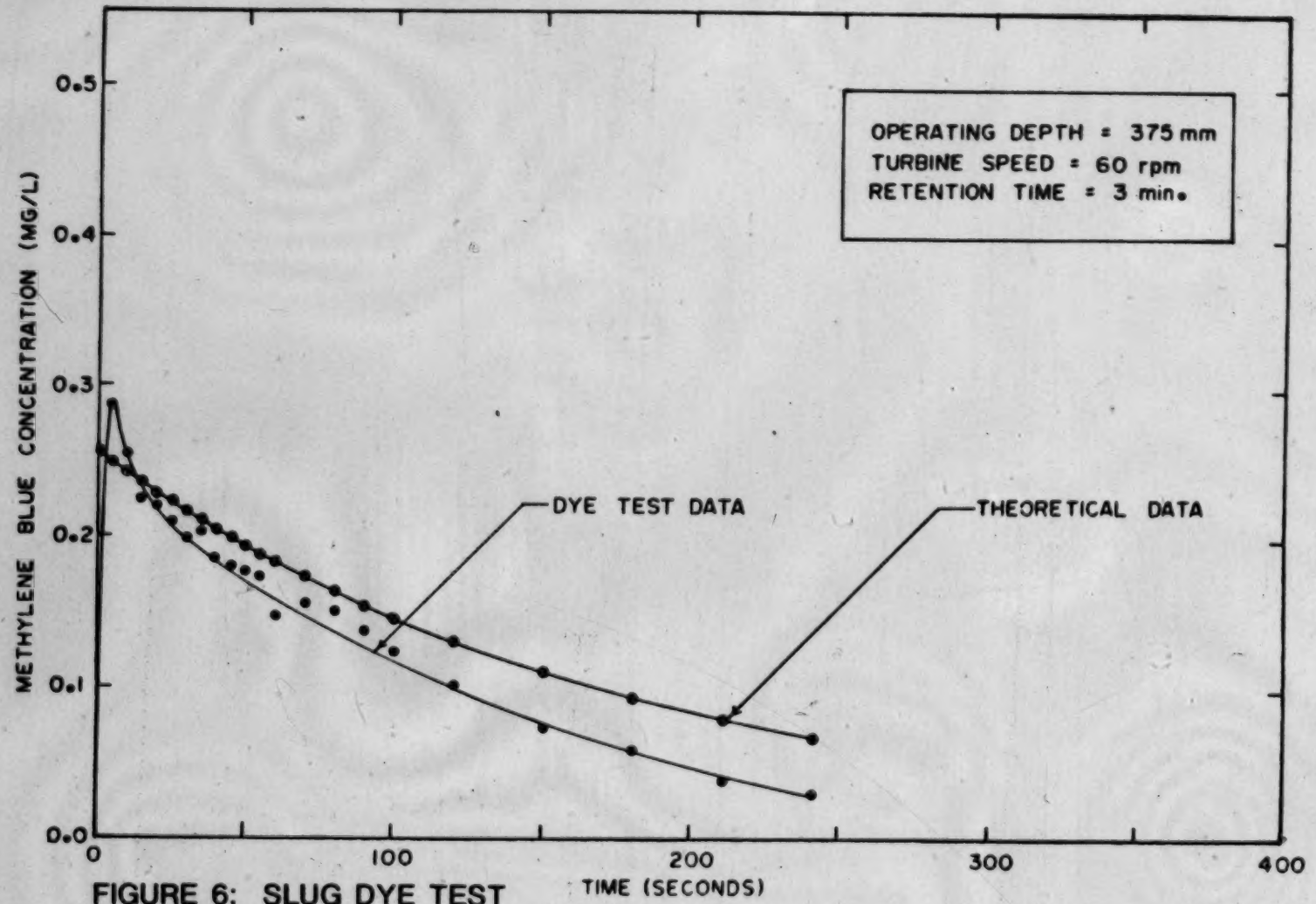

FIGURE 6: SLUG DYE TEST

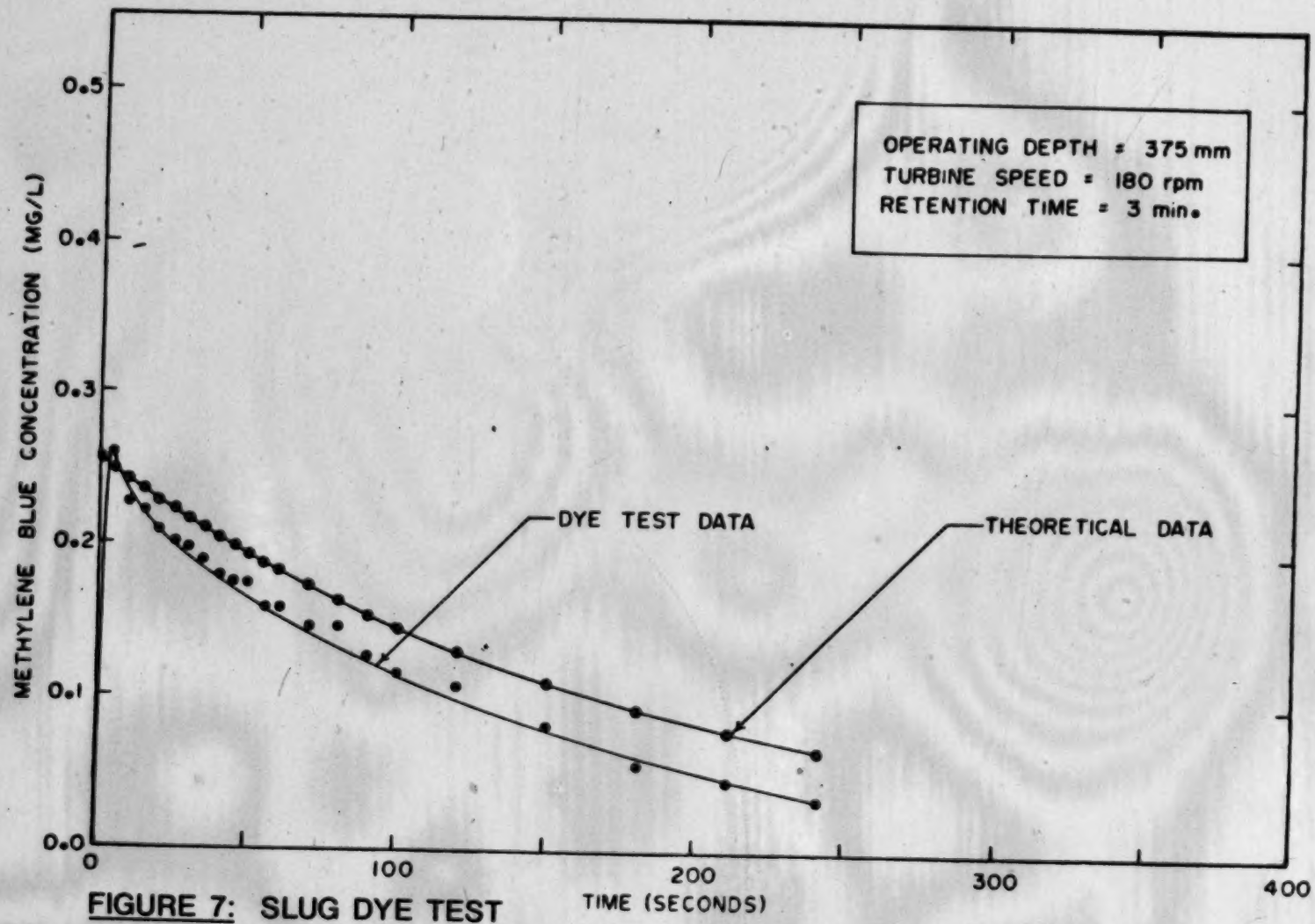

FIGURE 7: SLUG DYE TEST

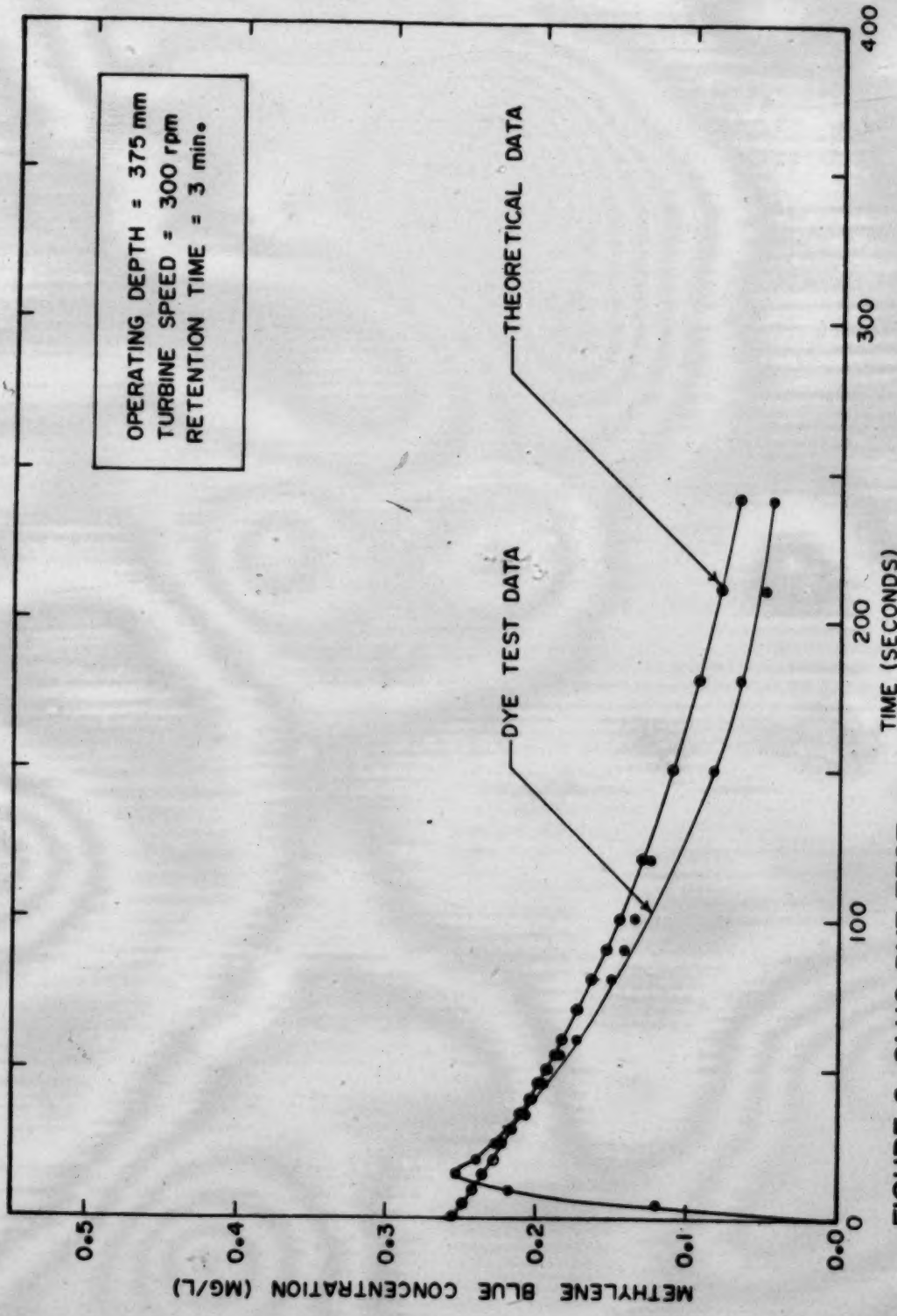

FIGURE 8: SLUG DYE TEST

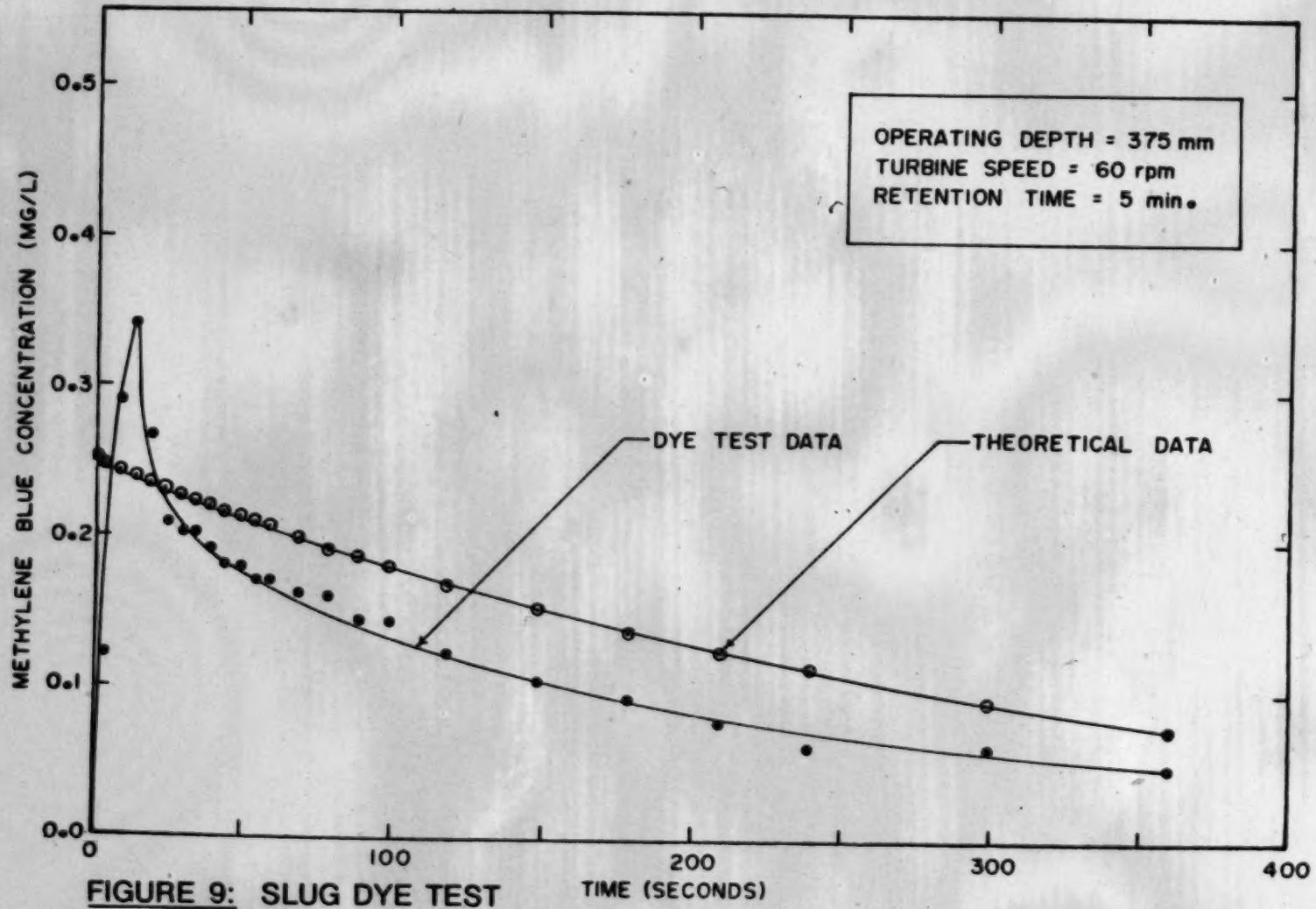

FIGURE 9: SLUG DYE TEST

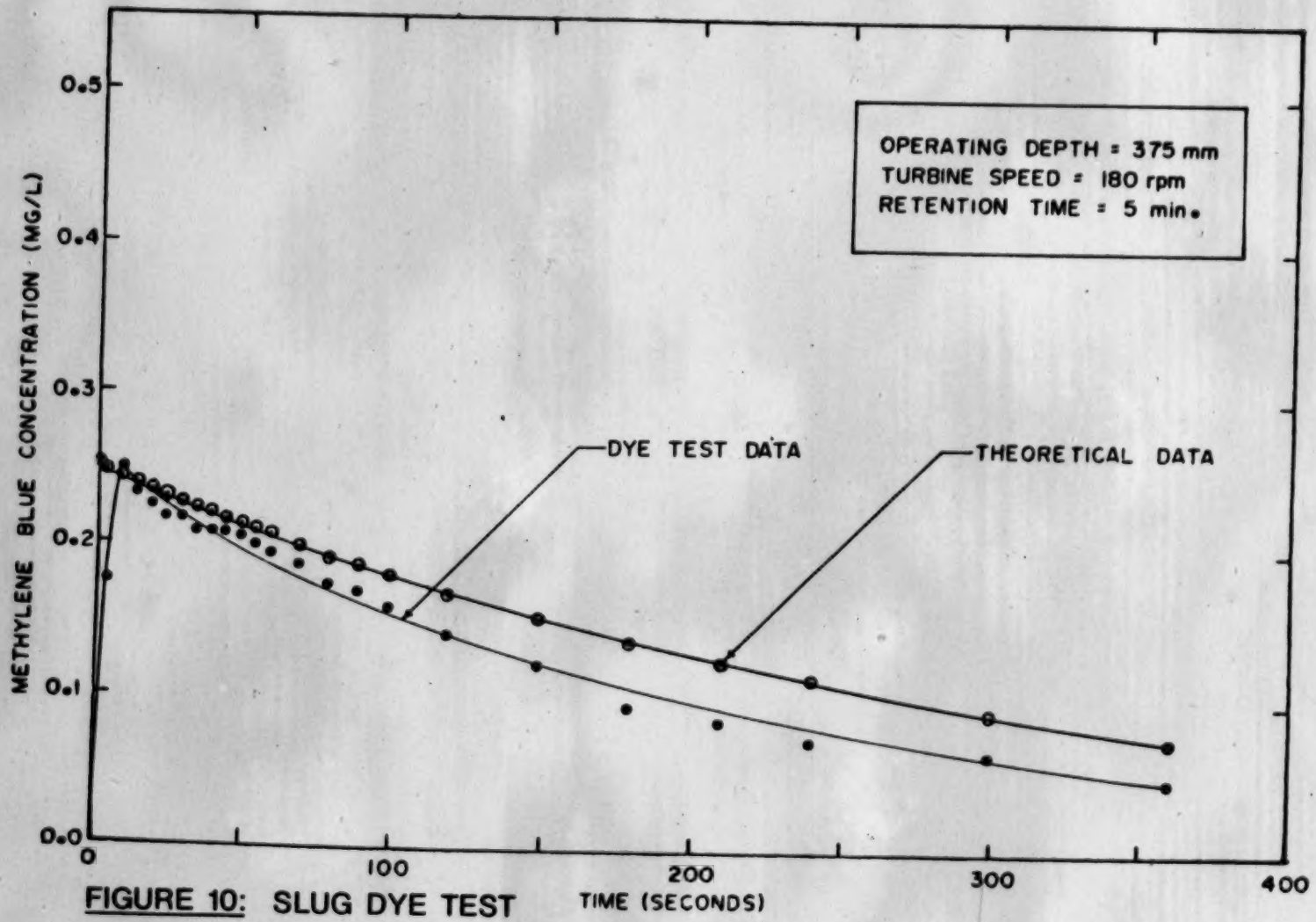

FIGURE 10: SLUG DYE TEST

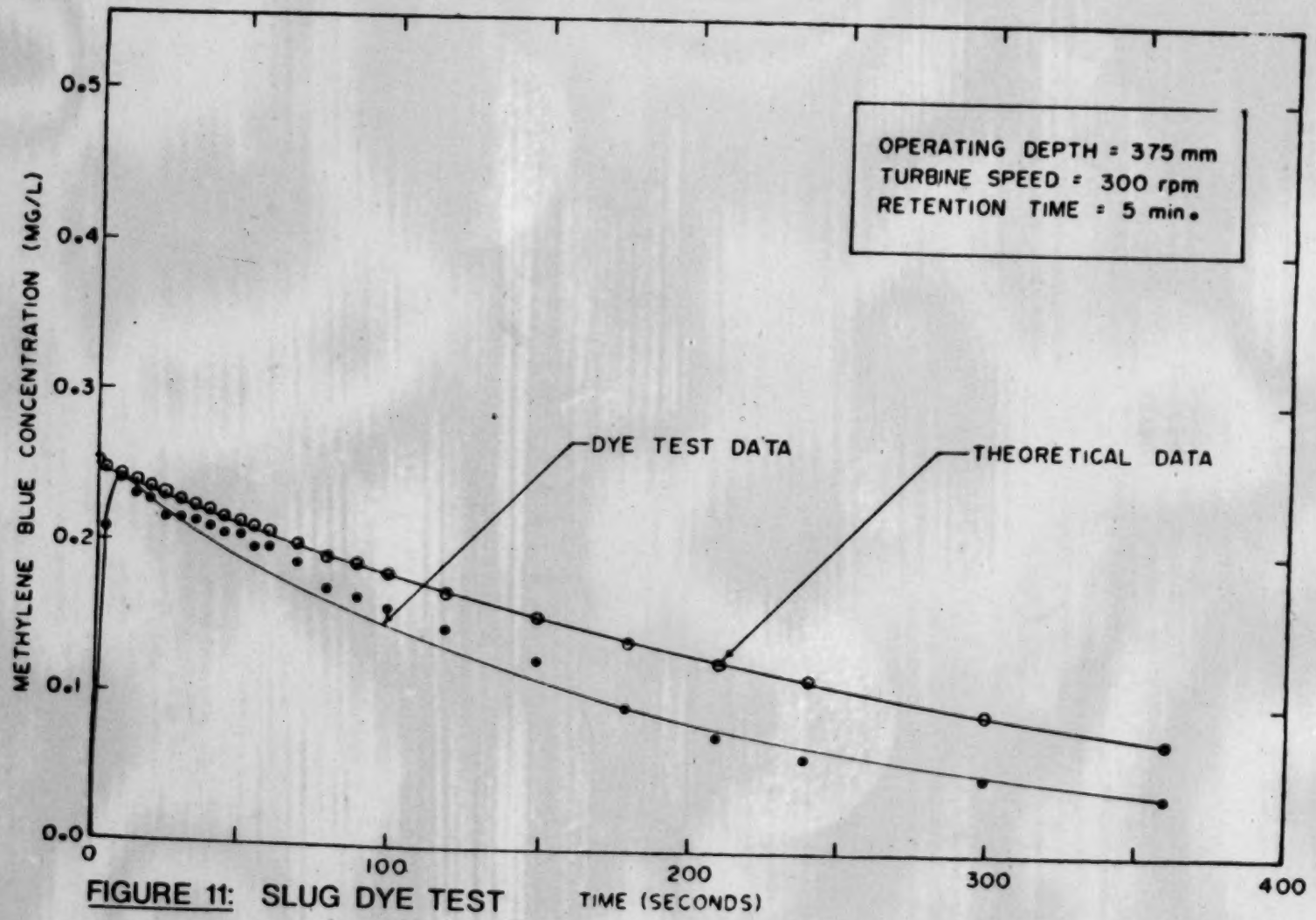

FIGURE 11: SLUG DYE TEST

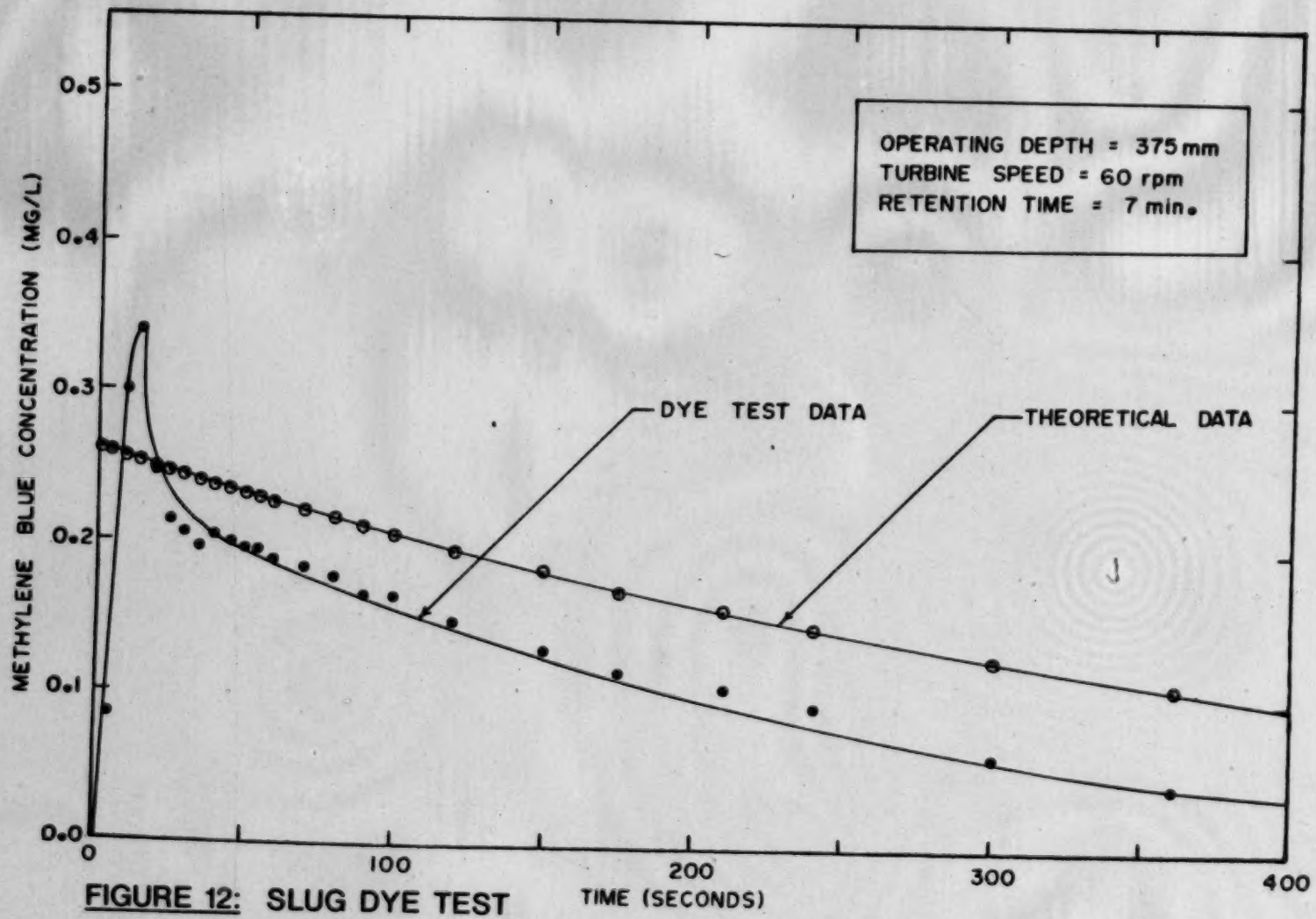

FIGURE 12: SLUG DYE TEST

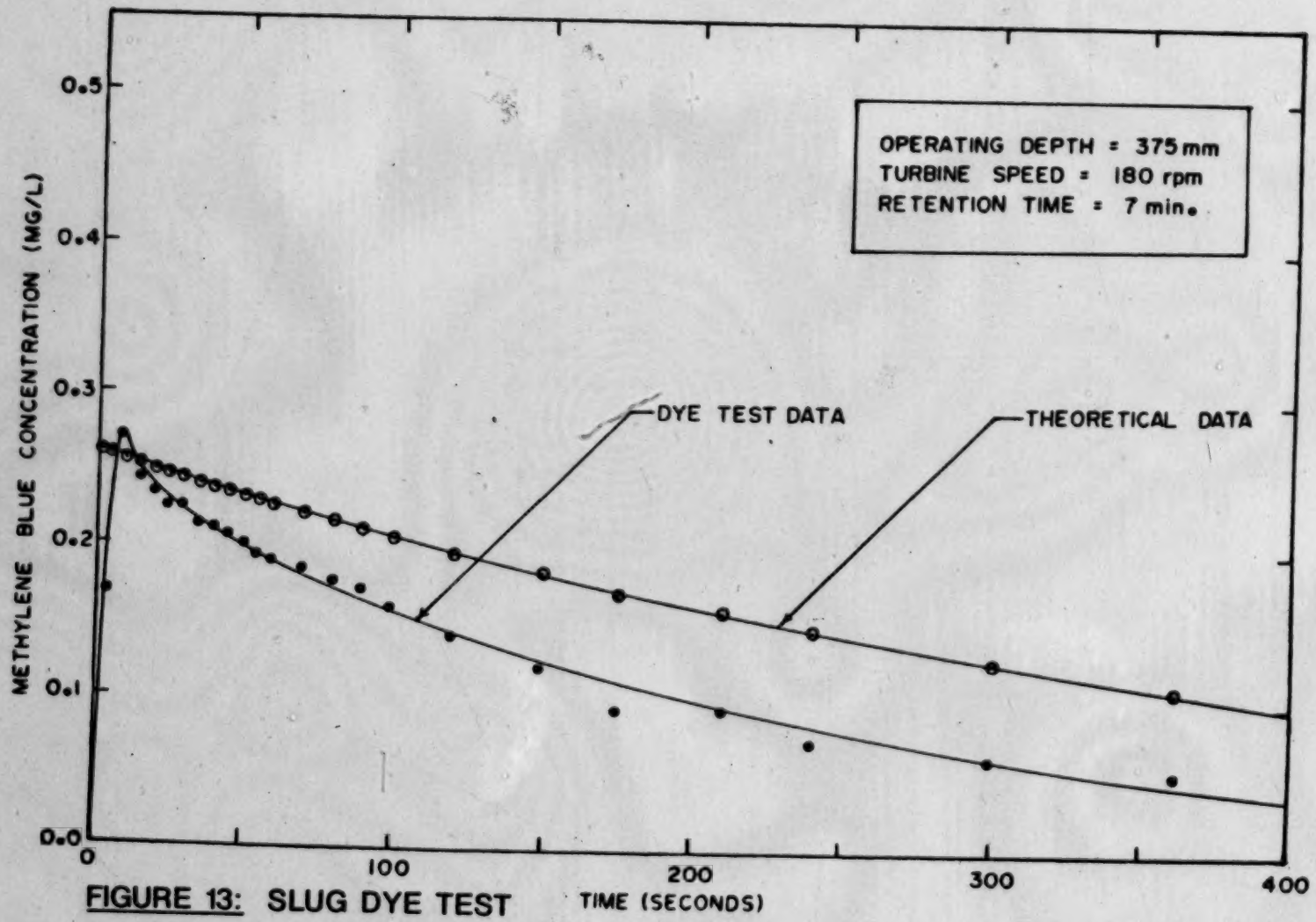

FIGURE 13: SLUG DYE TEST

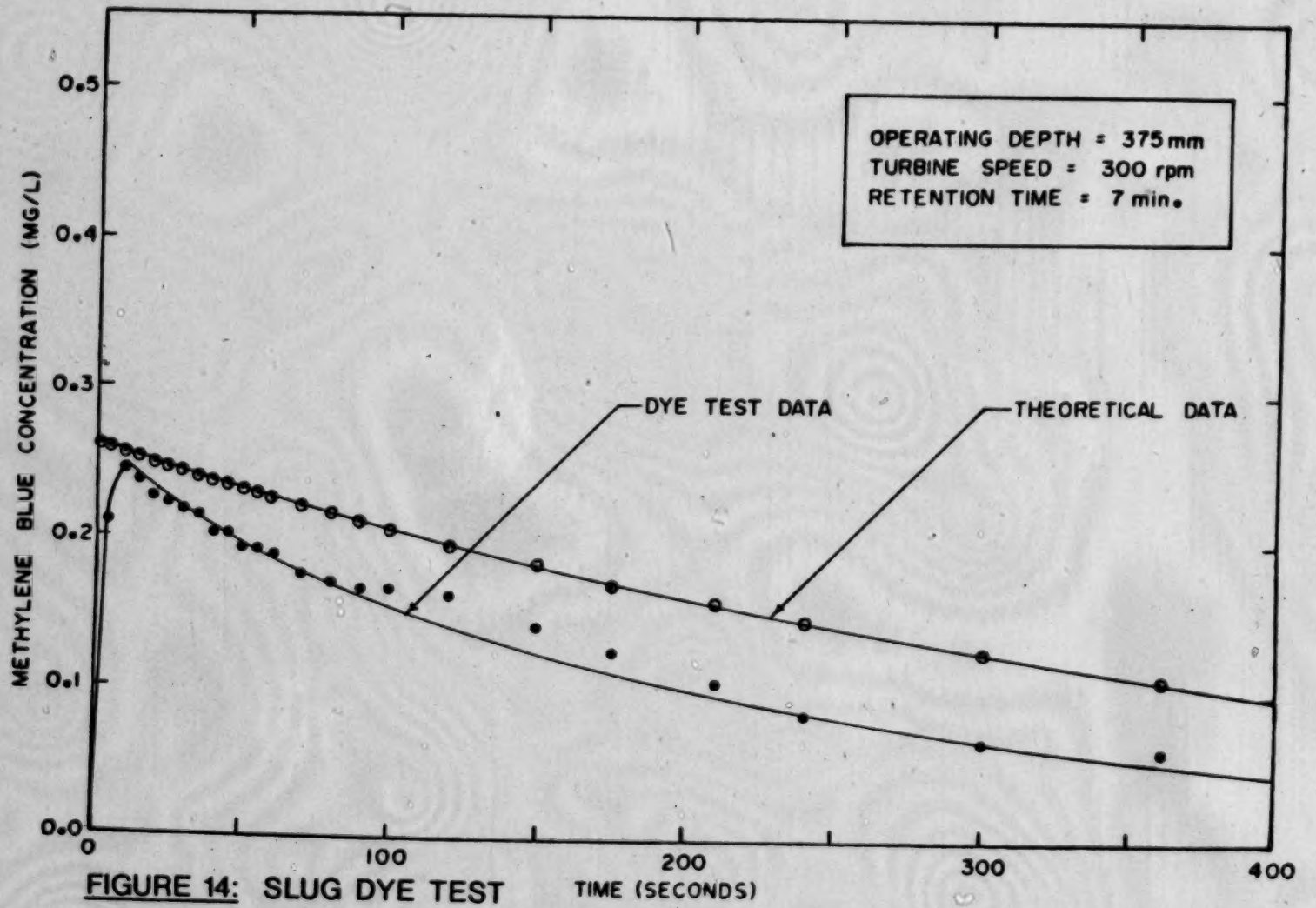

FIGURE 14: SLUG DYE TEST

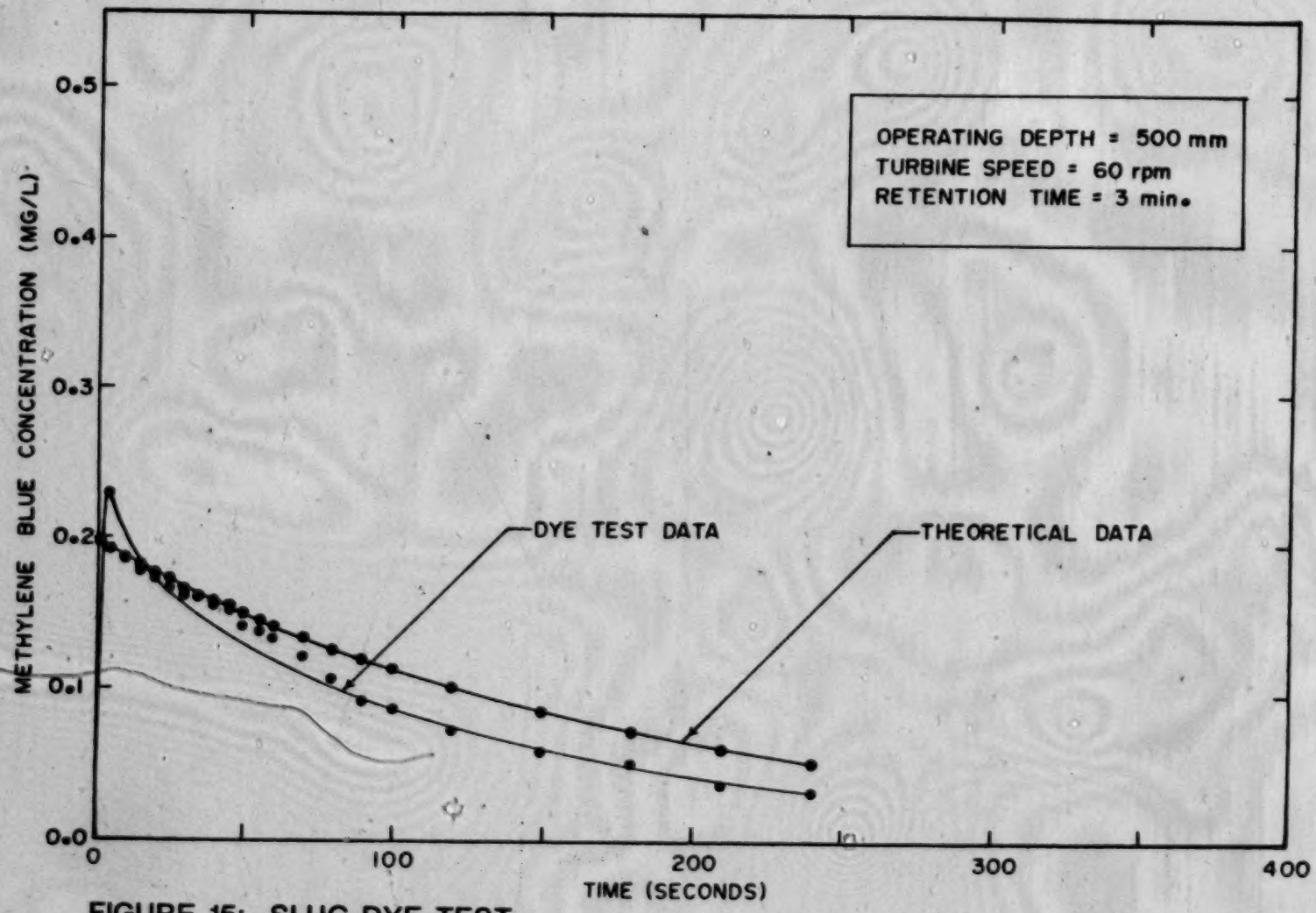

FIGURE 15: SLUG DYE TEST

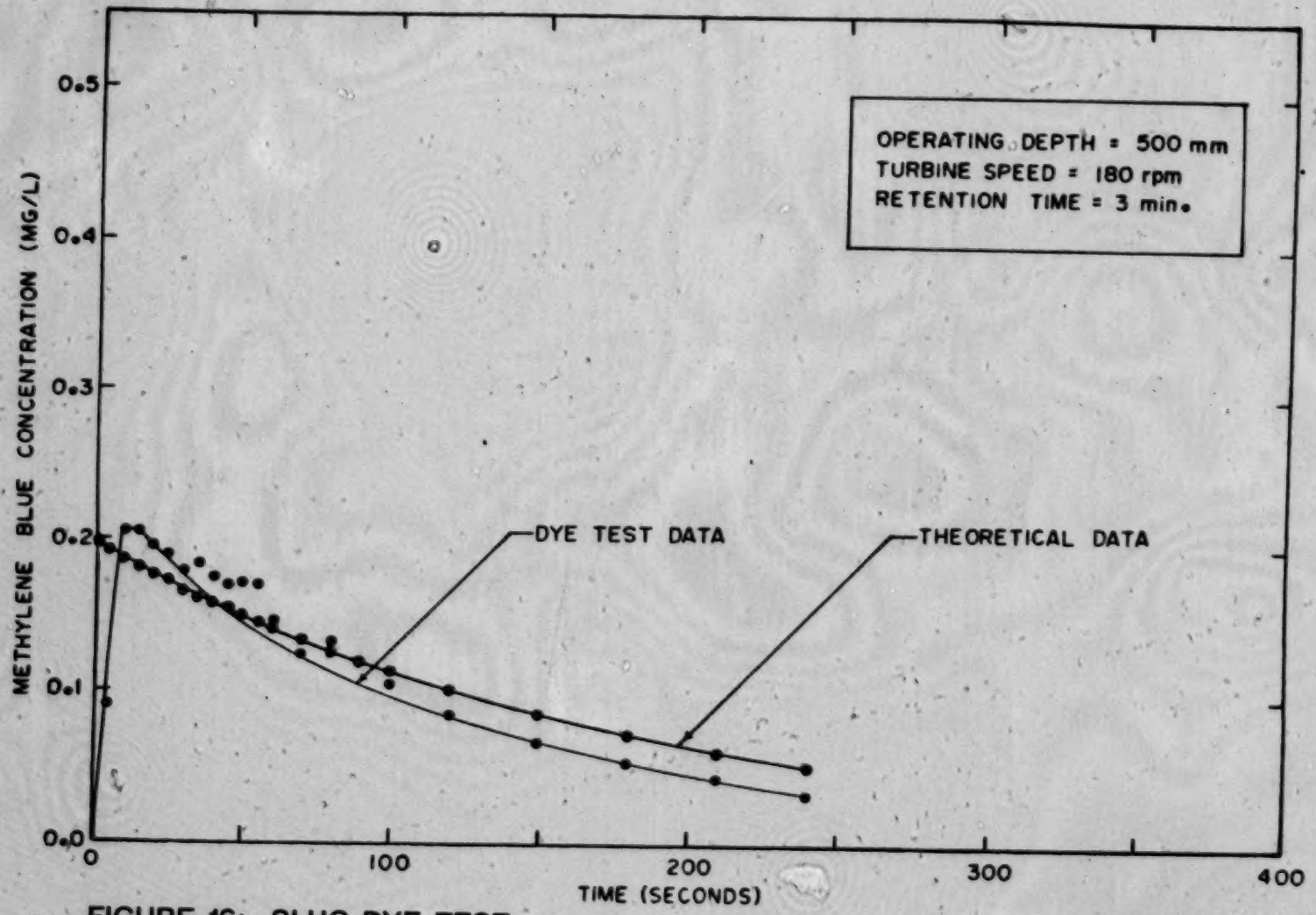

FIGURE 16: SLUG DYE TEST

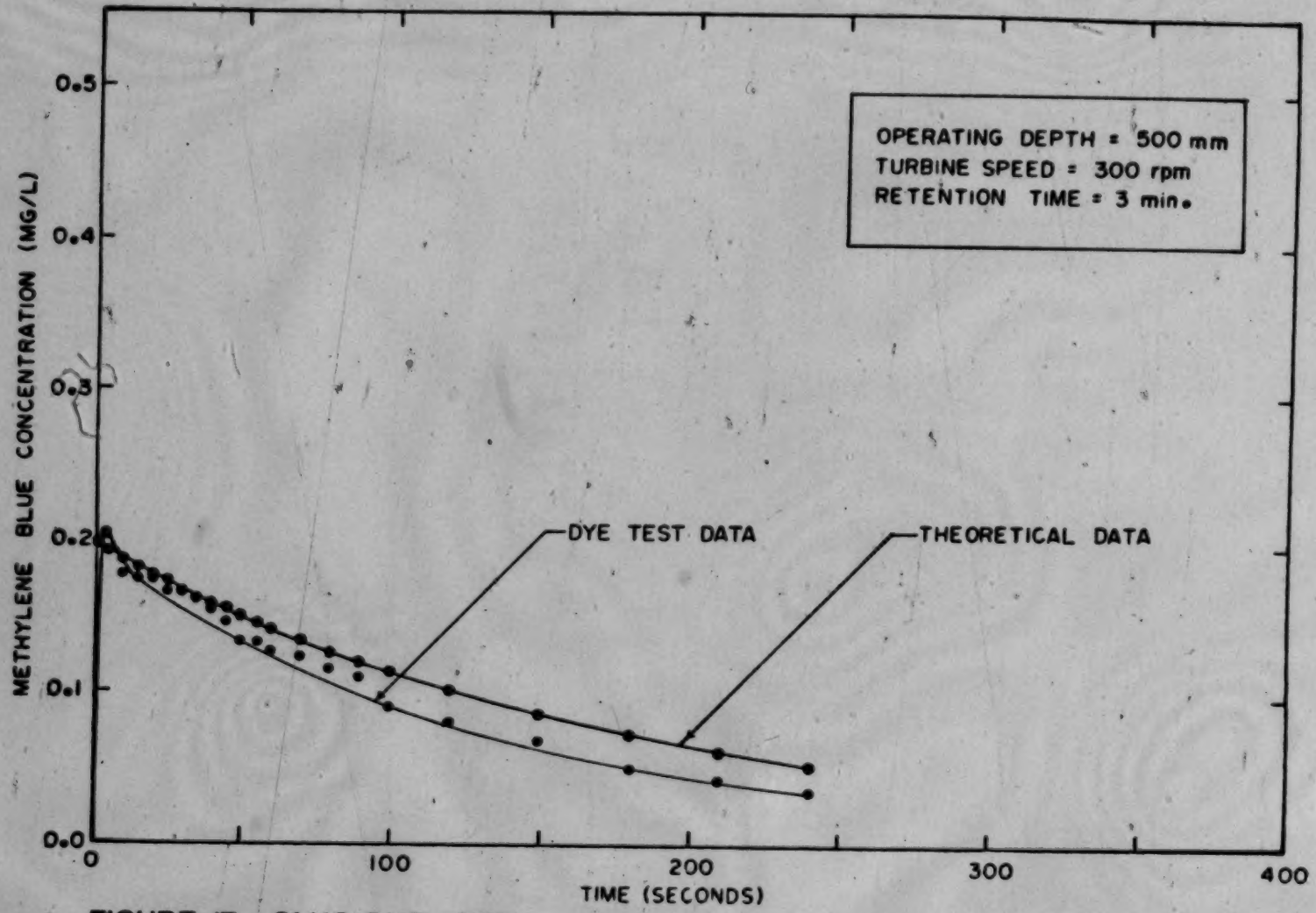

FIGURE 17: SLUG DYE TEST

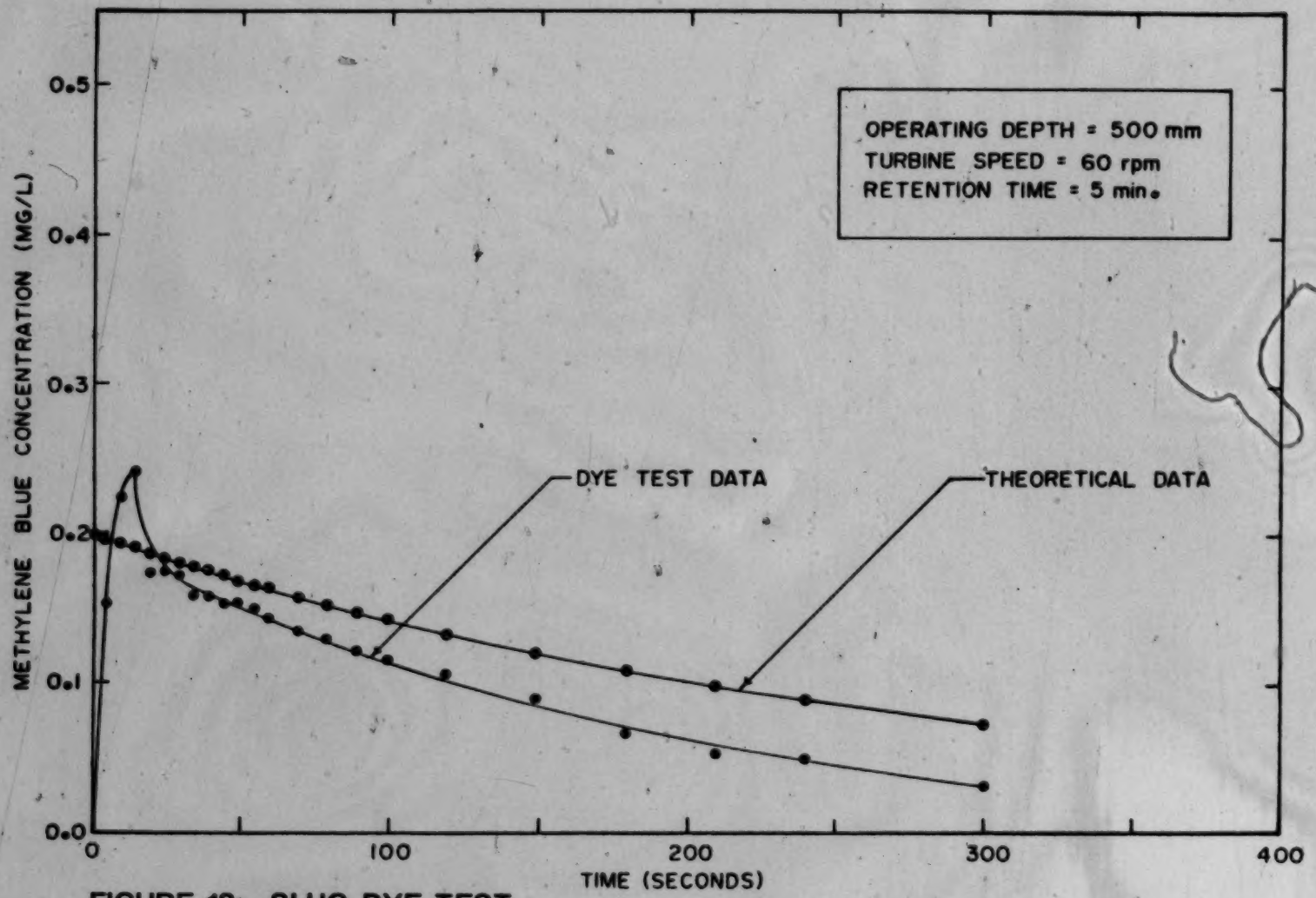

FIGURE 18: SLUG DYE TEST

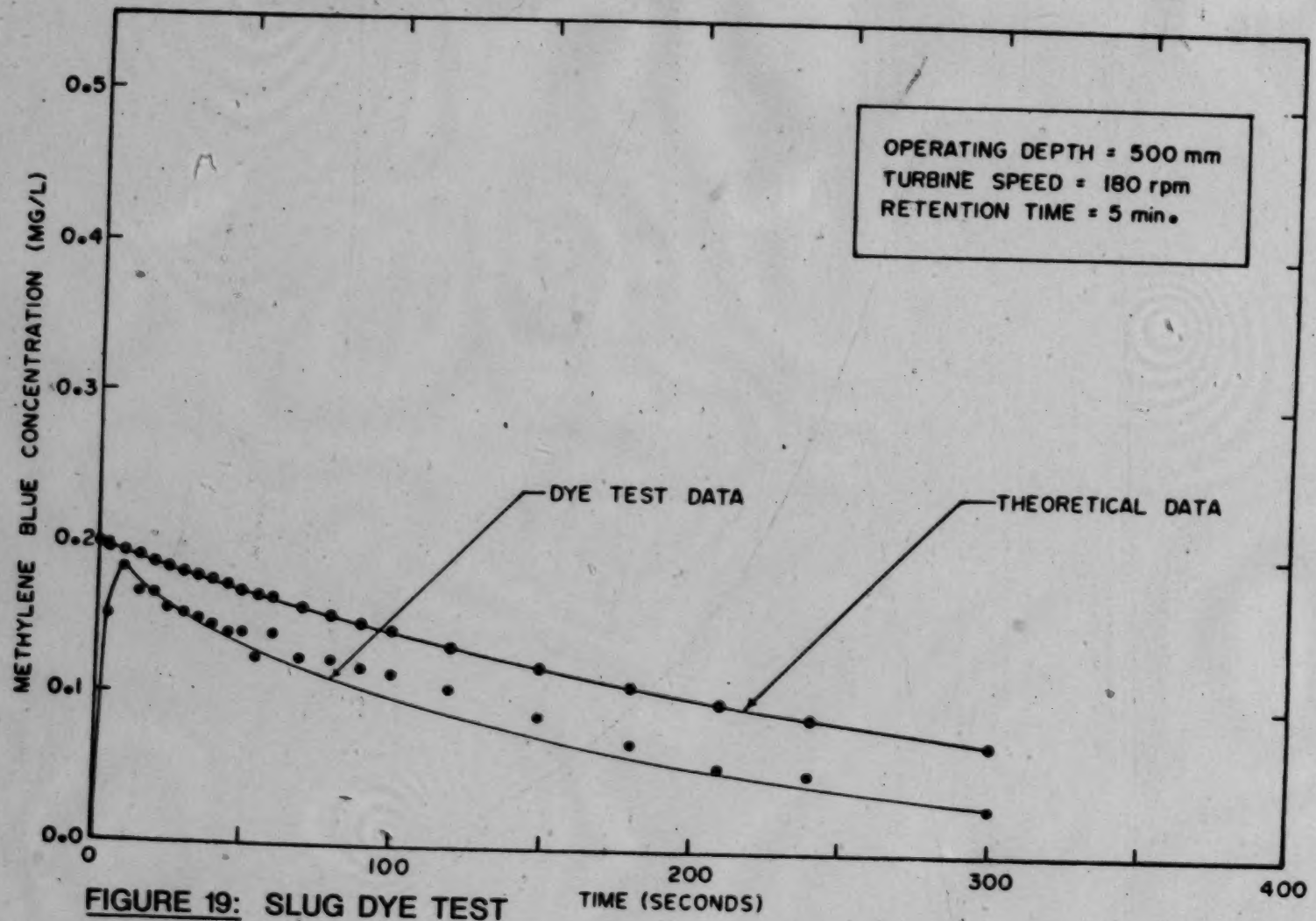

FIGURE 19: SLUG DYE TEST

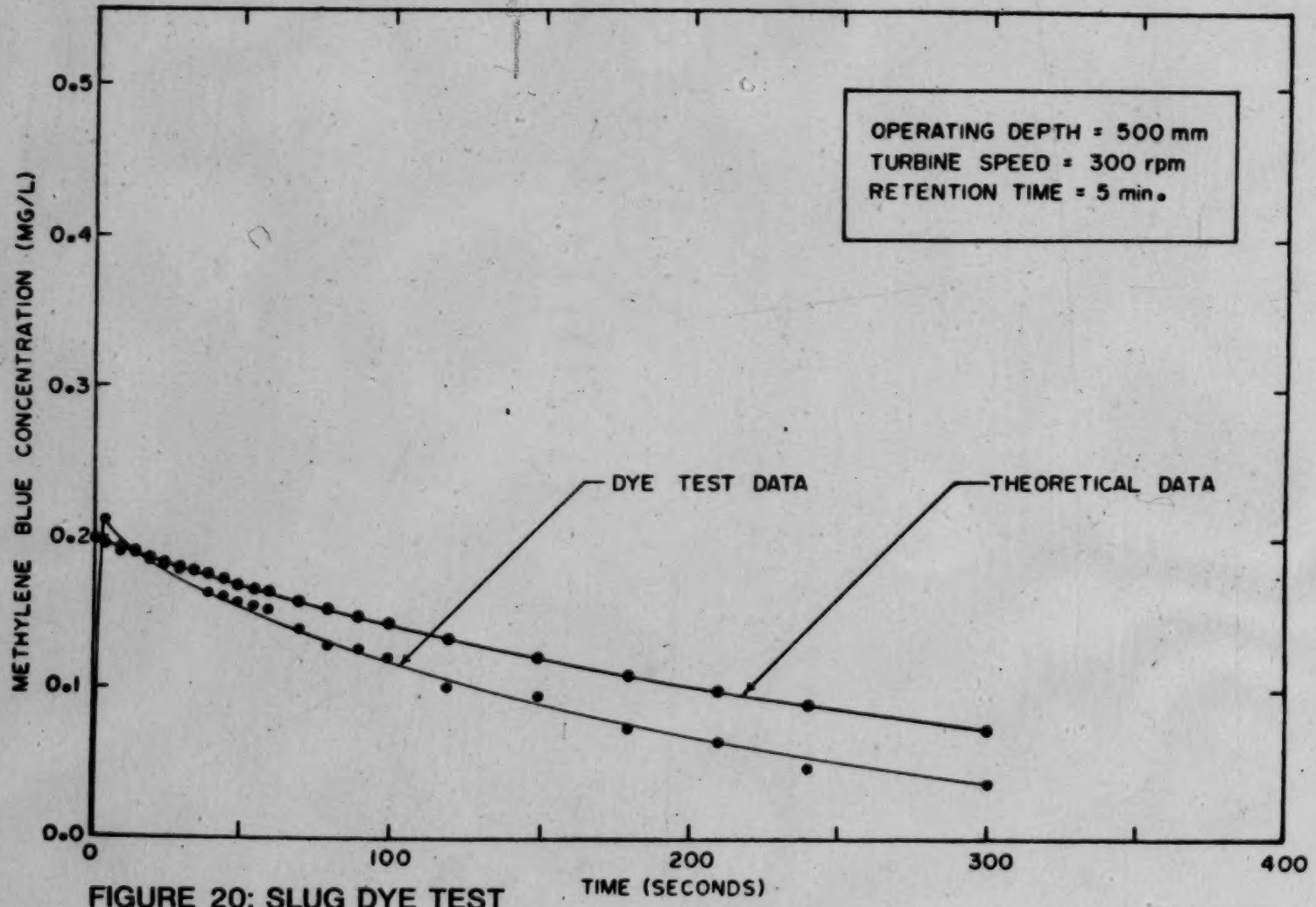

FIGURE 20: SLUG DYE TEST

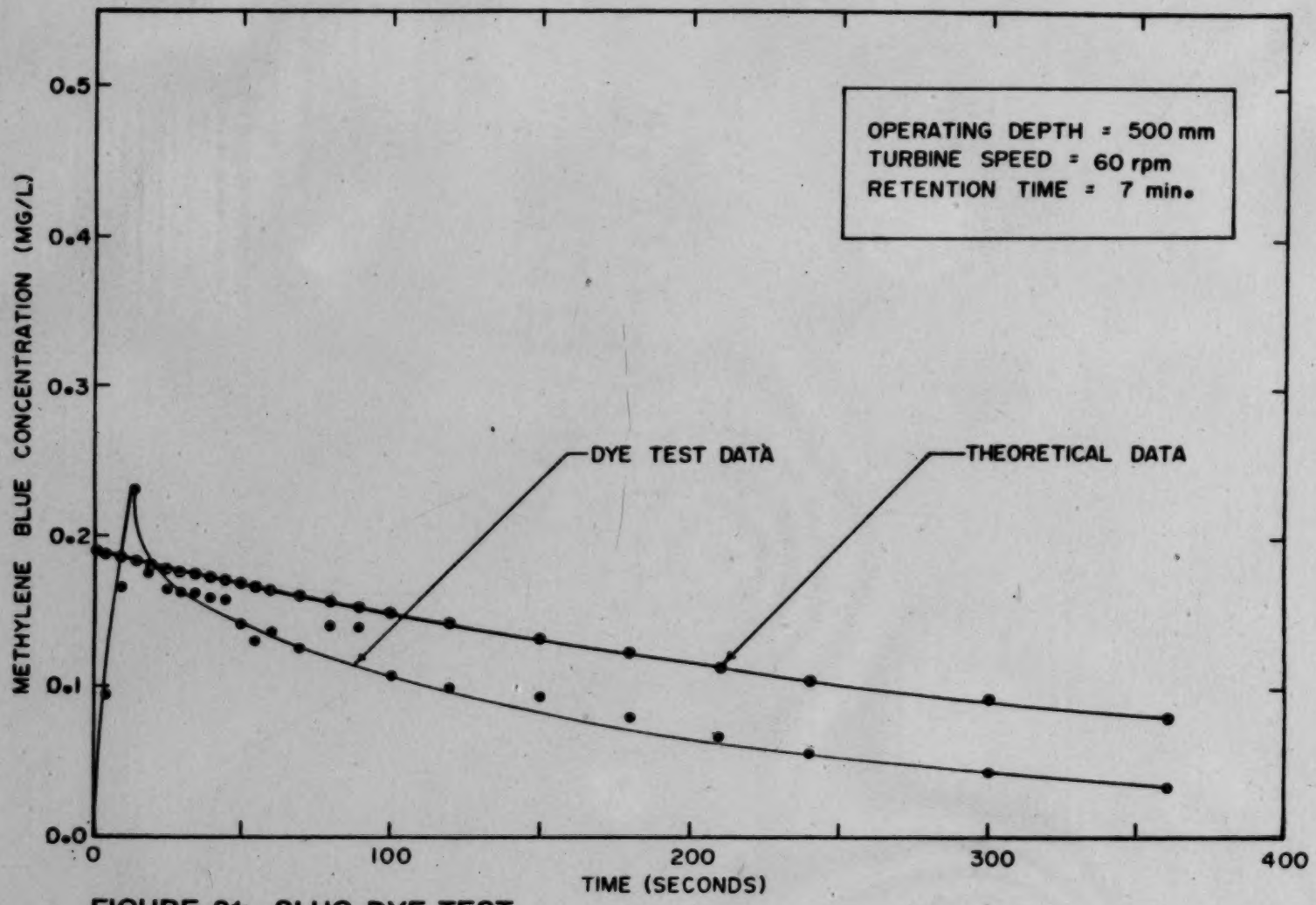

FIGURE 21: SLUG DYE TEST

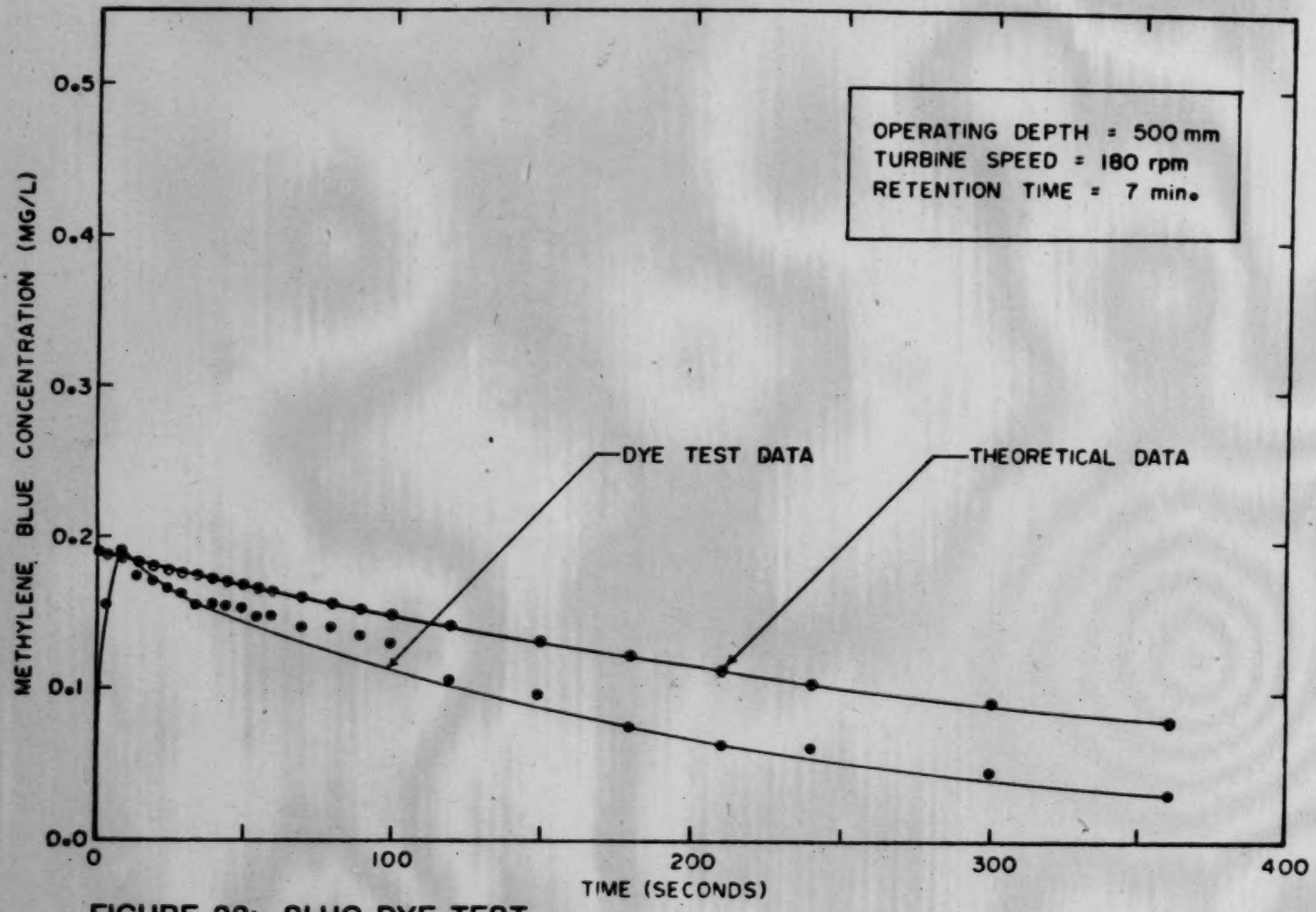

FIGURE 22: SLUG DYE TEST

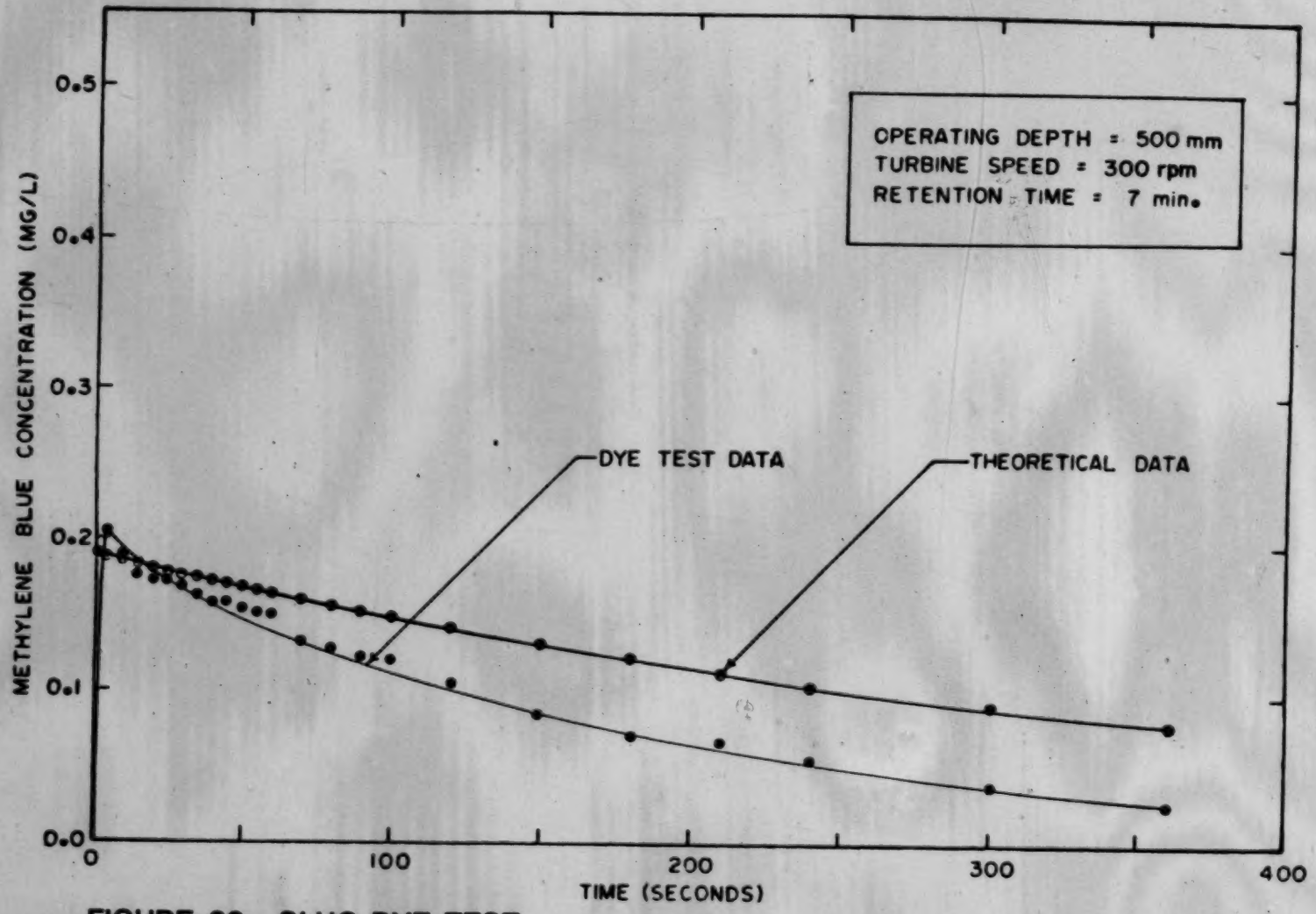

FIGURE 23: SLUG DYE TEST

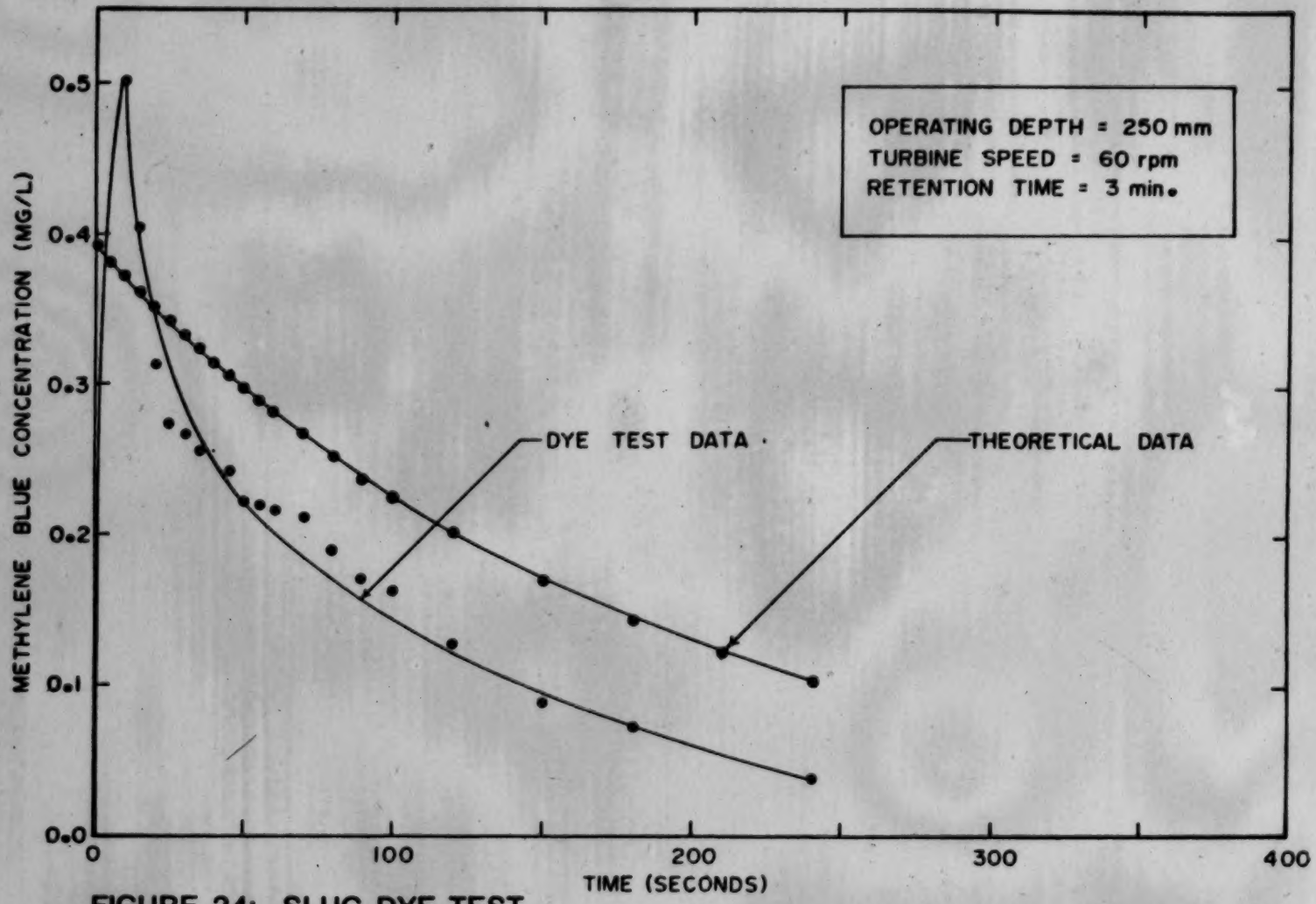

FIGURE 24: SLUG DYE TEST

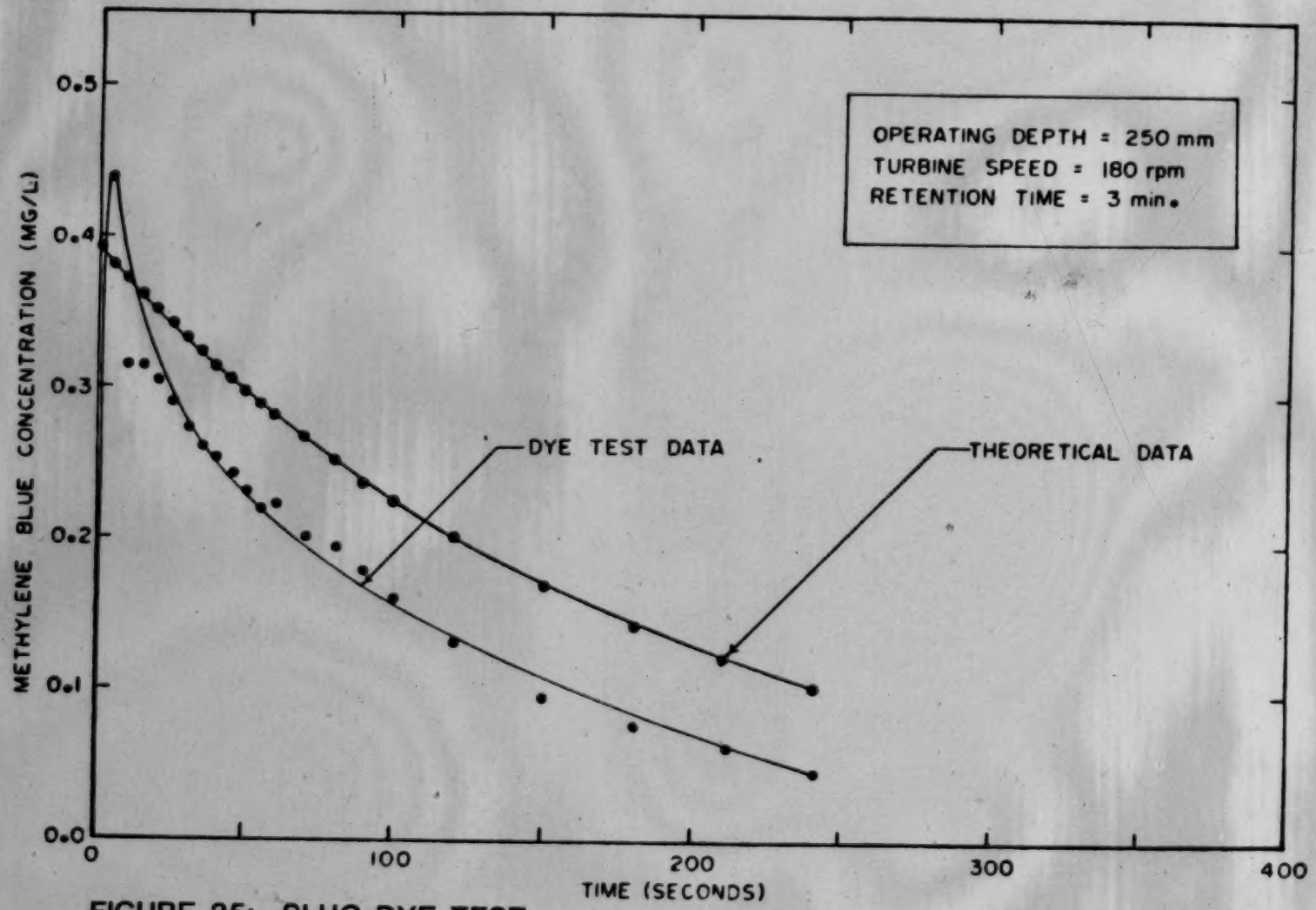

FIGURE 25: SLUG DYE TEST

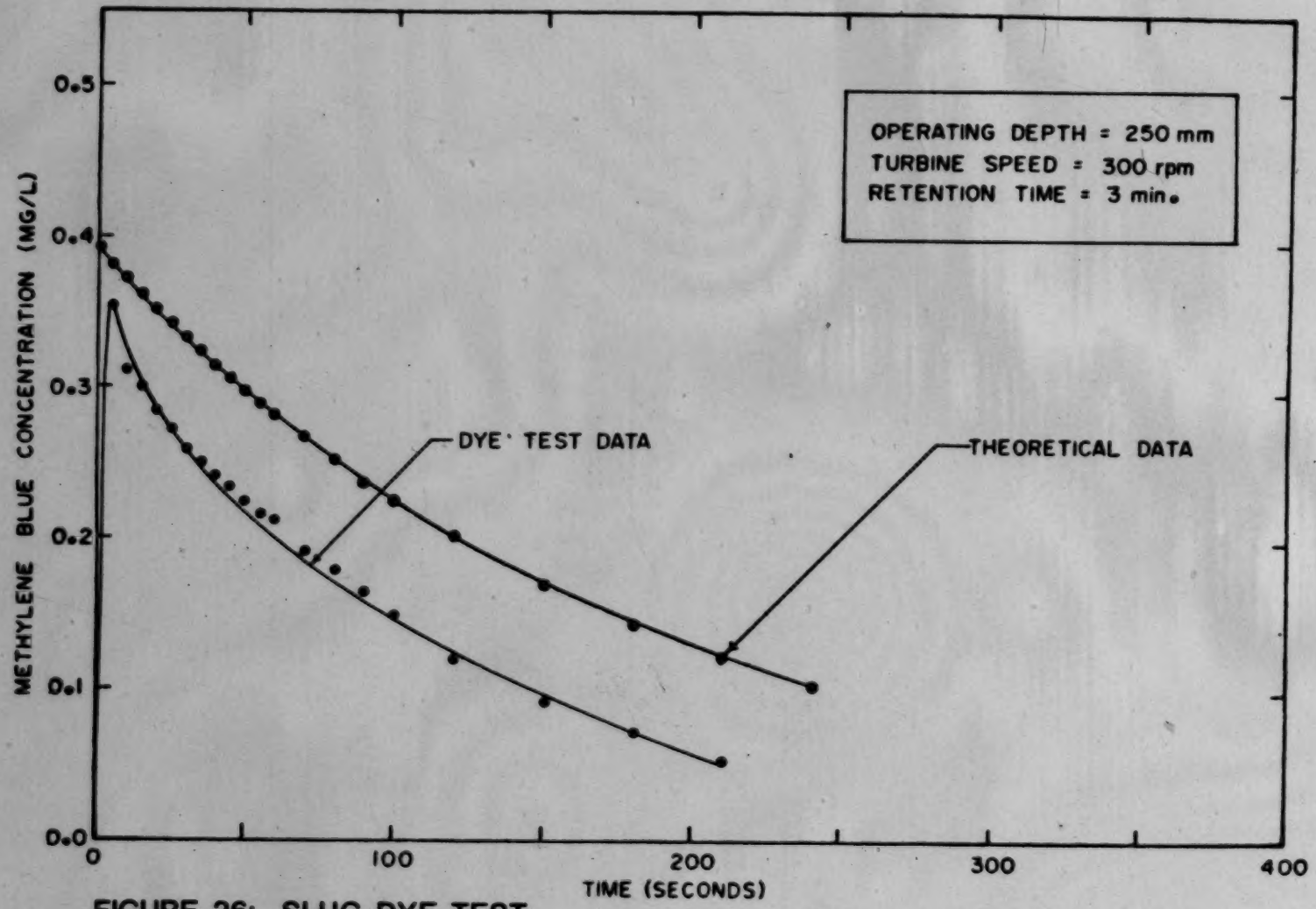

FIGURE 26: SLUG DYE TEST

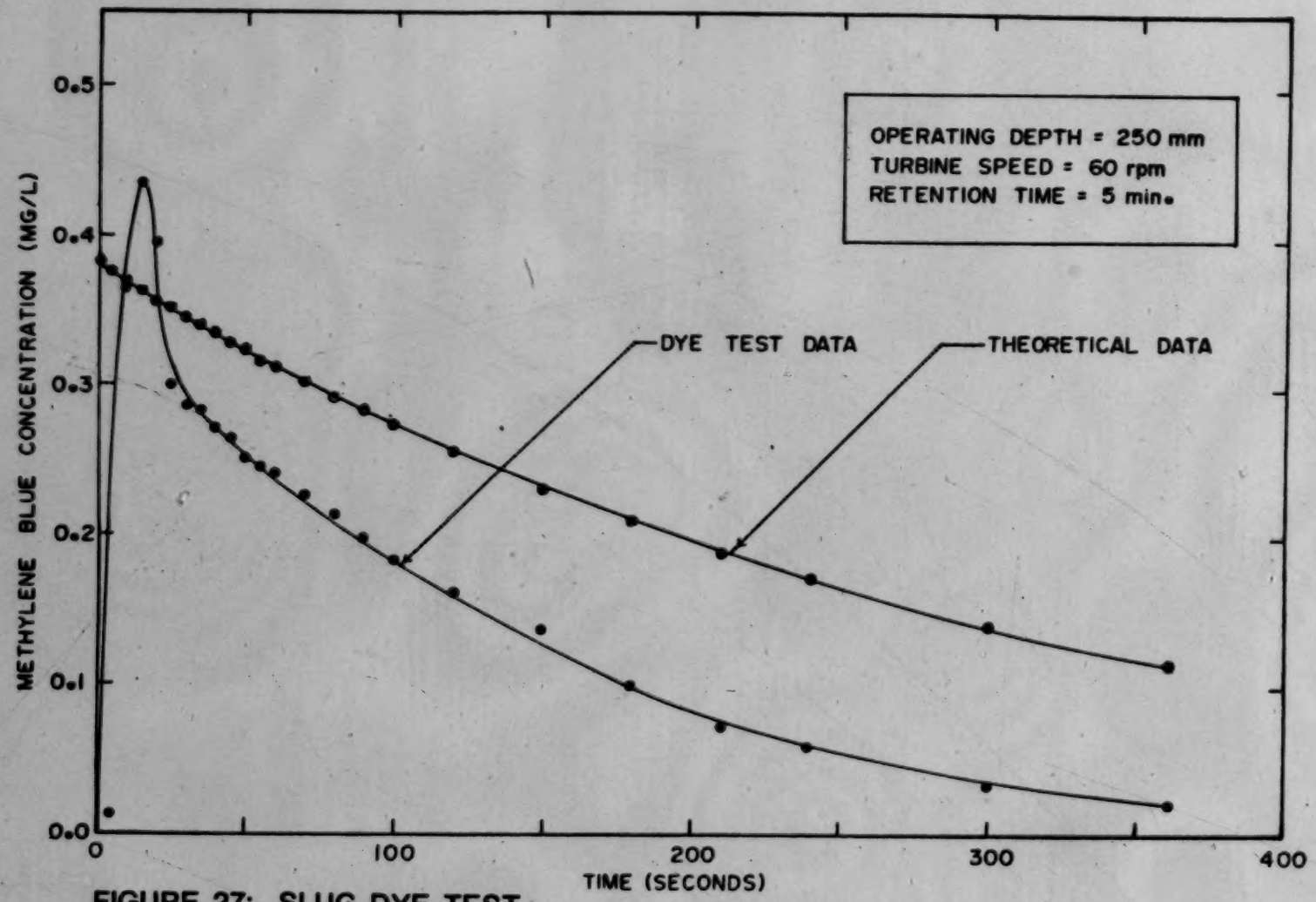

FIGURE 27: SLUG DYE TEST

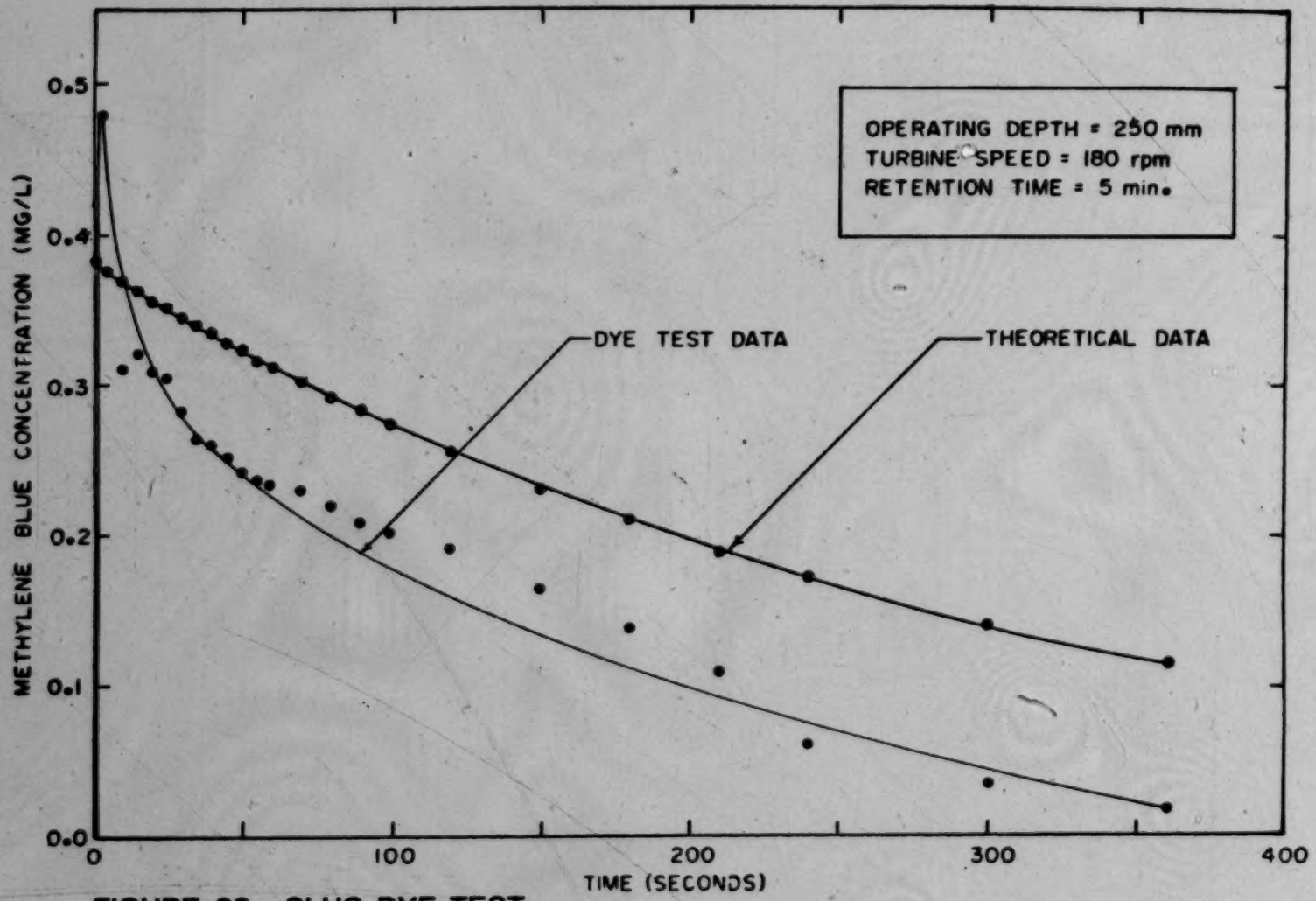

FIGURE 28: SLUG DYE TEST

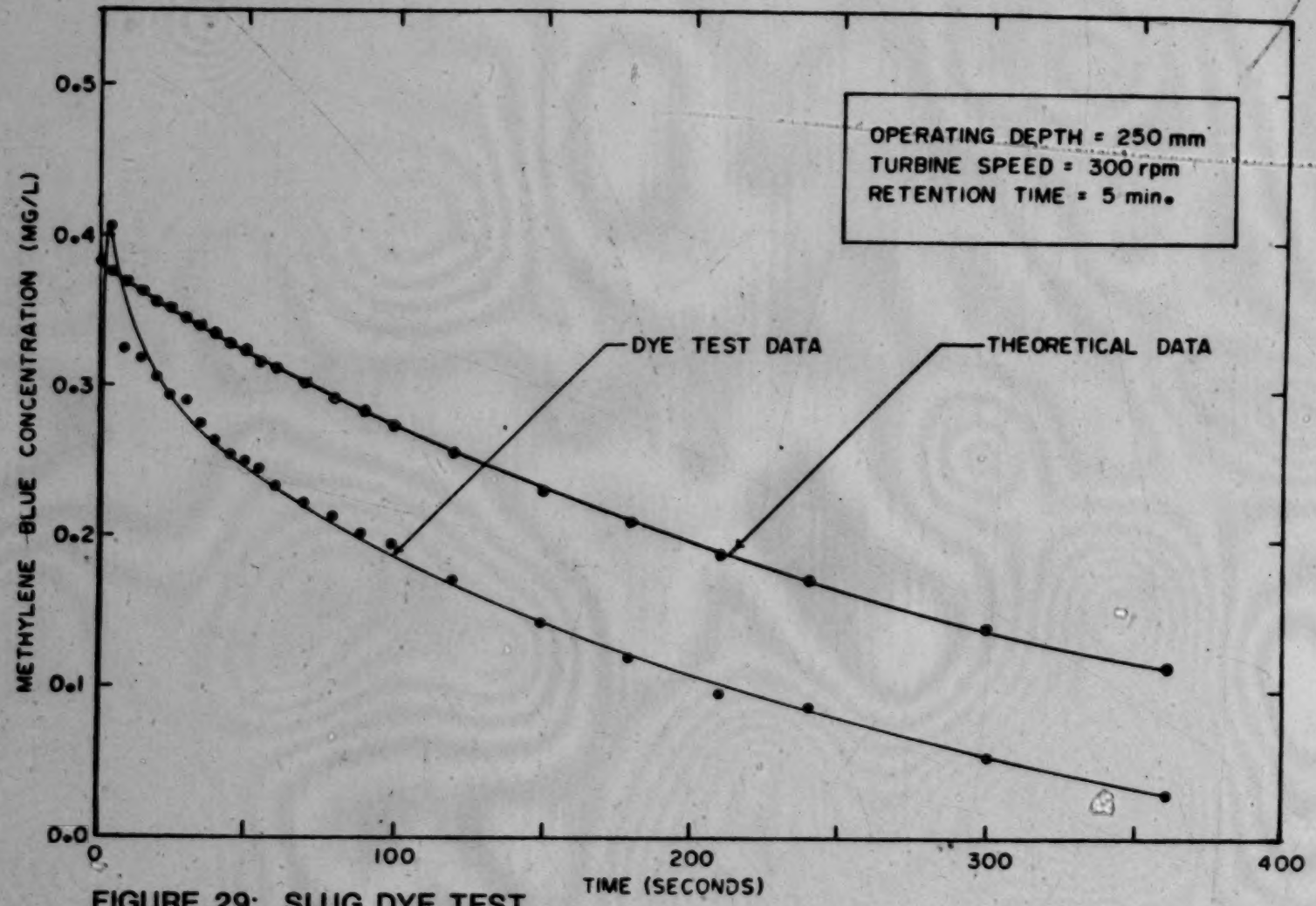

FIGURE 29: SLUG DYE TEST

TABLE 11 CONTROL TEST RESULTS FOR TOTAL COLIFORMS

Test	N_o (counts/100 ml)	N (counts/100 ml)	$\log_{10}(N/N_o)$	Retention Time (min.)	QG/QL
1	7.8 E06	6.2 E06	-0.102	5.05	1.26
2	1.3 E07	1.1 E07	-0.071	7.80	2.56
3	1.4 E07	1.3 E07	-0.031	3.55	1.13
4	8.3 E06	5.5 E06	-0.181	10.1	2.56

N_o - counts/100 ml prior to aeration with oxygen

N - counts/100 ml following aeration with oxygen

$\frac{N}{N_o}$ - survival ratio

$\frac{QG}{QL}$ - oxygen: primary effluent flowrate ratio.

TABLE 12 CONTROL TEST RESULTS FOR FECAL COLIFORMS

Test	N_o (counts/100 ml)	N (counts/100 ml)	$\log_{10}(N/N_o)$	Retention Time (min.)	QG/QL
1	8.0 E05	7.8 E05	-0.009	5.05	1.26
2	9.5 E05	9.1 E05	-0.018	7.80	2.56
3	1.4 E06	1.3 E06	-0.032	3.55	1.13
4	9.7 E05	8.5 E05	-0.056	10.1	2.56

N_o - counts/100 ml prior to aeration with oxygen

N - counts/100 ml following aeration with oxygen

$\frac{N}{N_o}$ - survival ratio

$\frac{QG}{QL}$ - oxygen: primary effluent flowrate ratio.

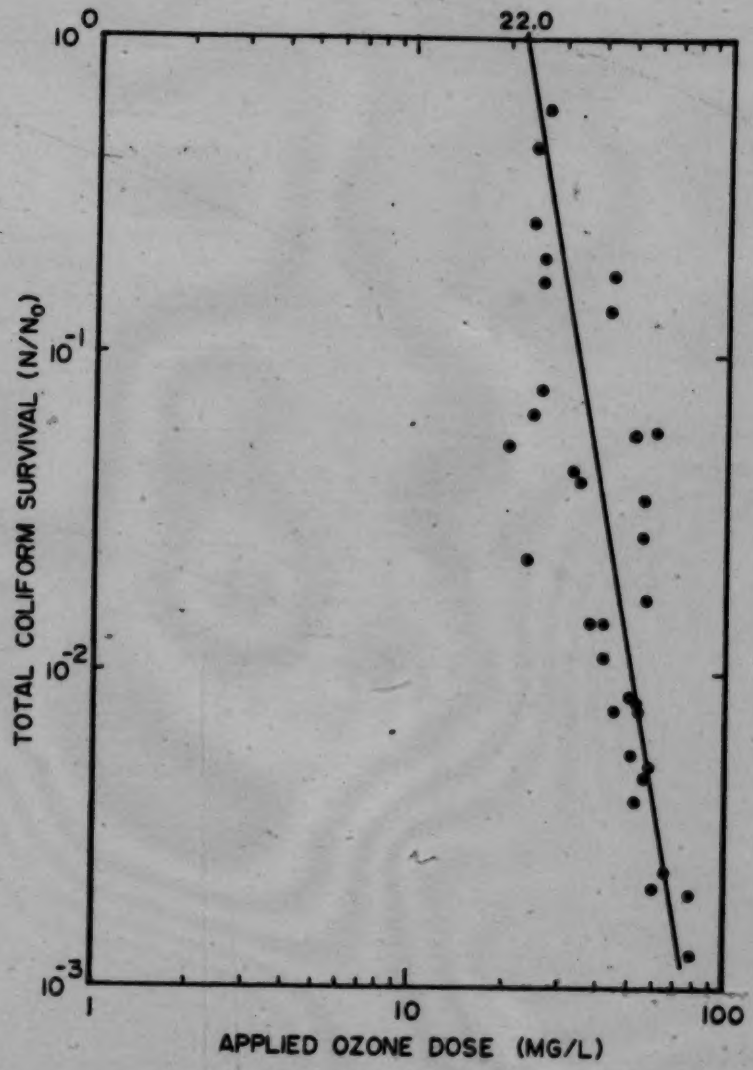

FIGURE 30: APPLIED OZONE DOSE VS. TOTAL COLIFORM SURVIVAL RATIO

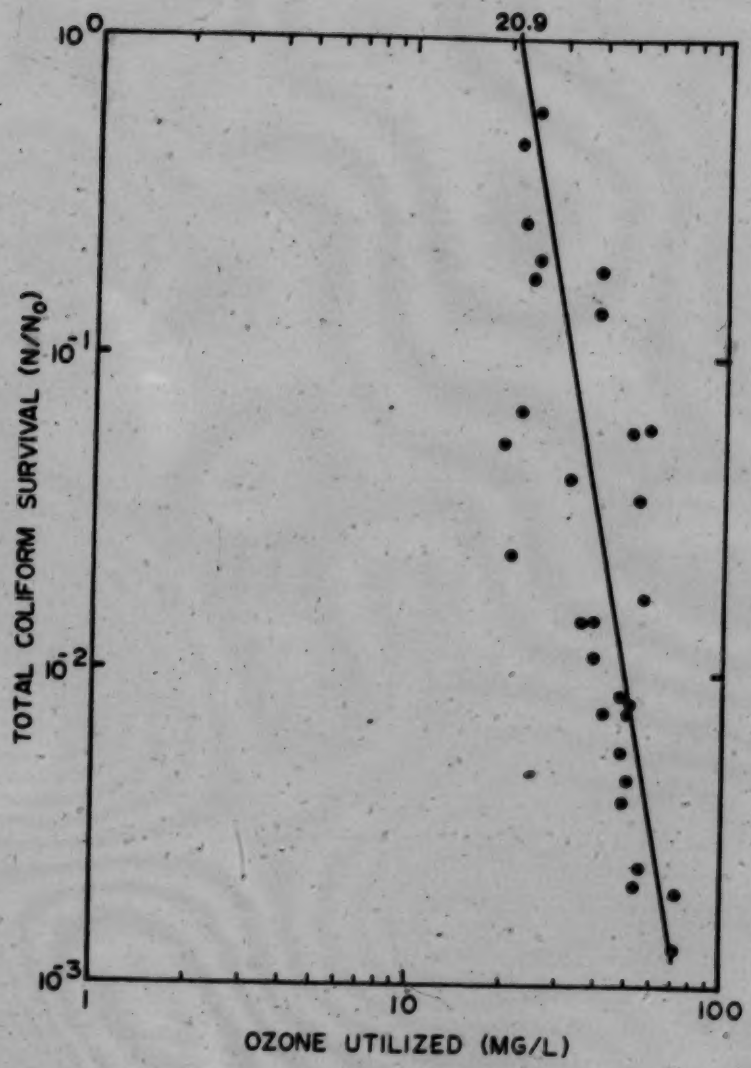

FIGURE 31: OZONE UTILIZED VS. TOTAL COLIFORM SURVIVAL RATIO

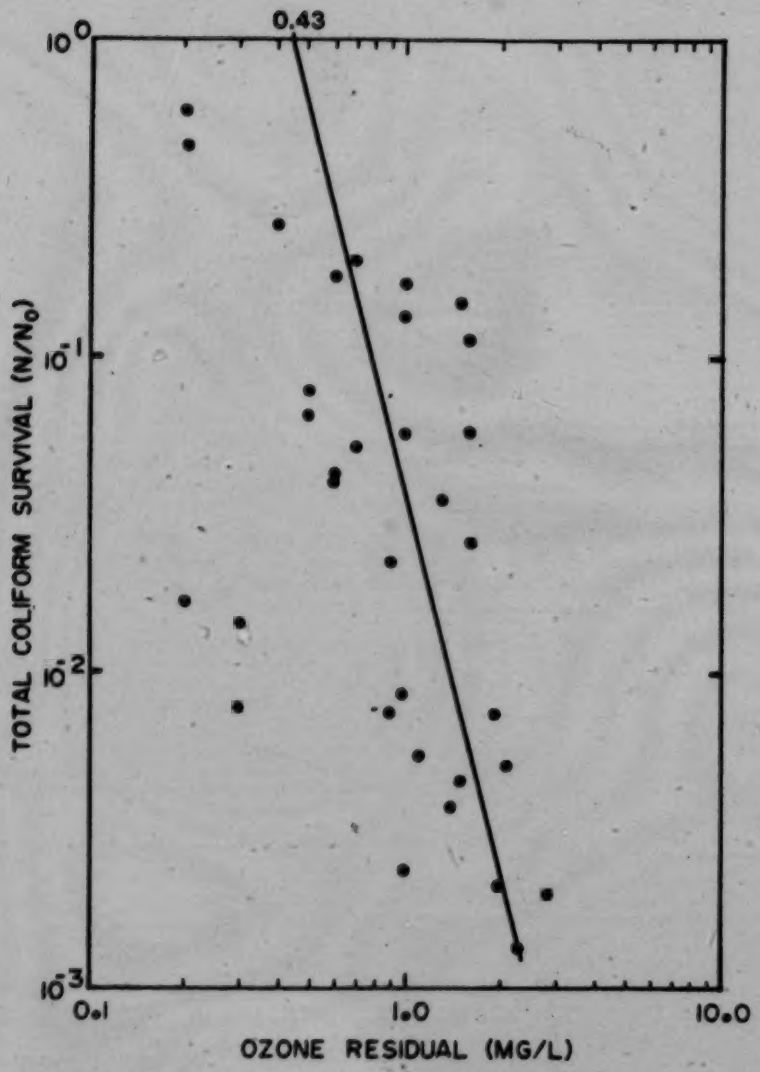

FIGURE 32: OZONE RESIDUAL VS. TOTAL COLIFORM SURVIVAL RATIO

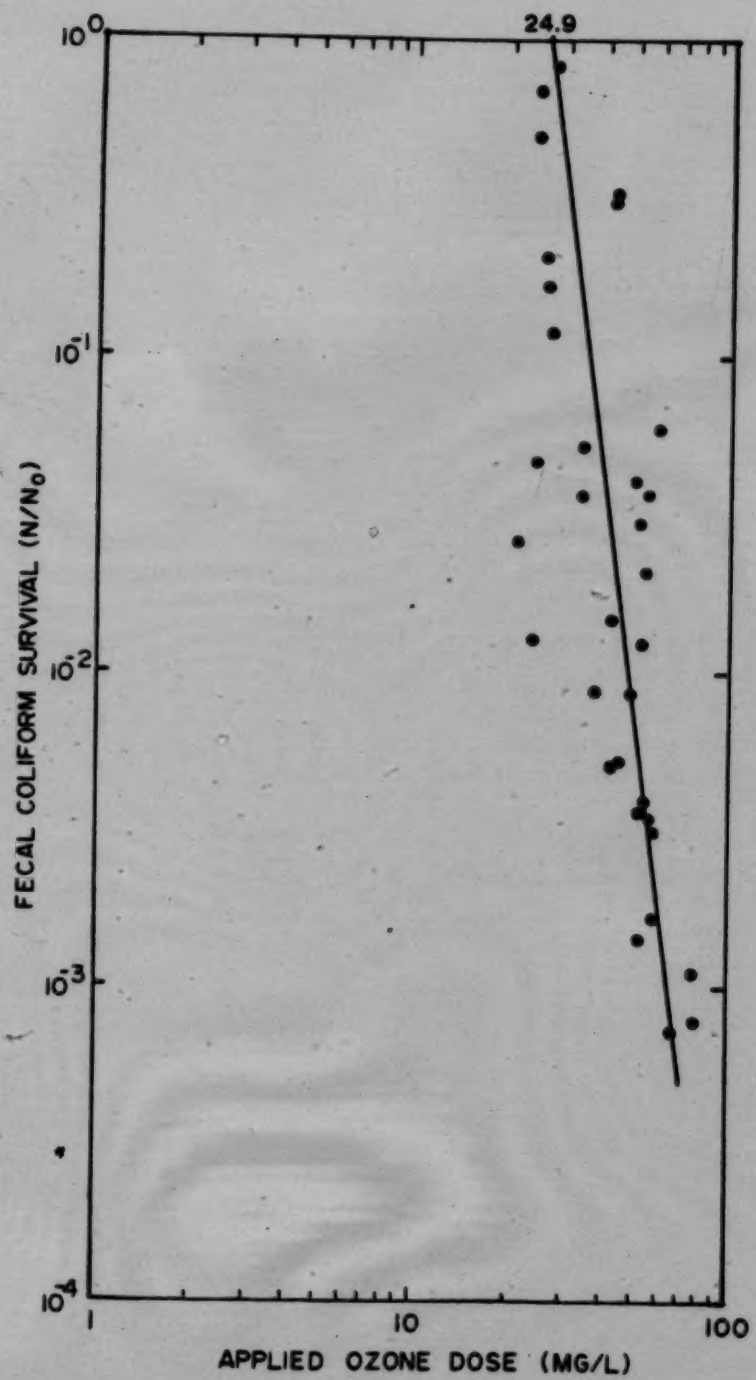

FIGURE 33: APPLIED OZONE DOSE VS. FECAL COLIFORM SURVIVAL RATIO

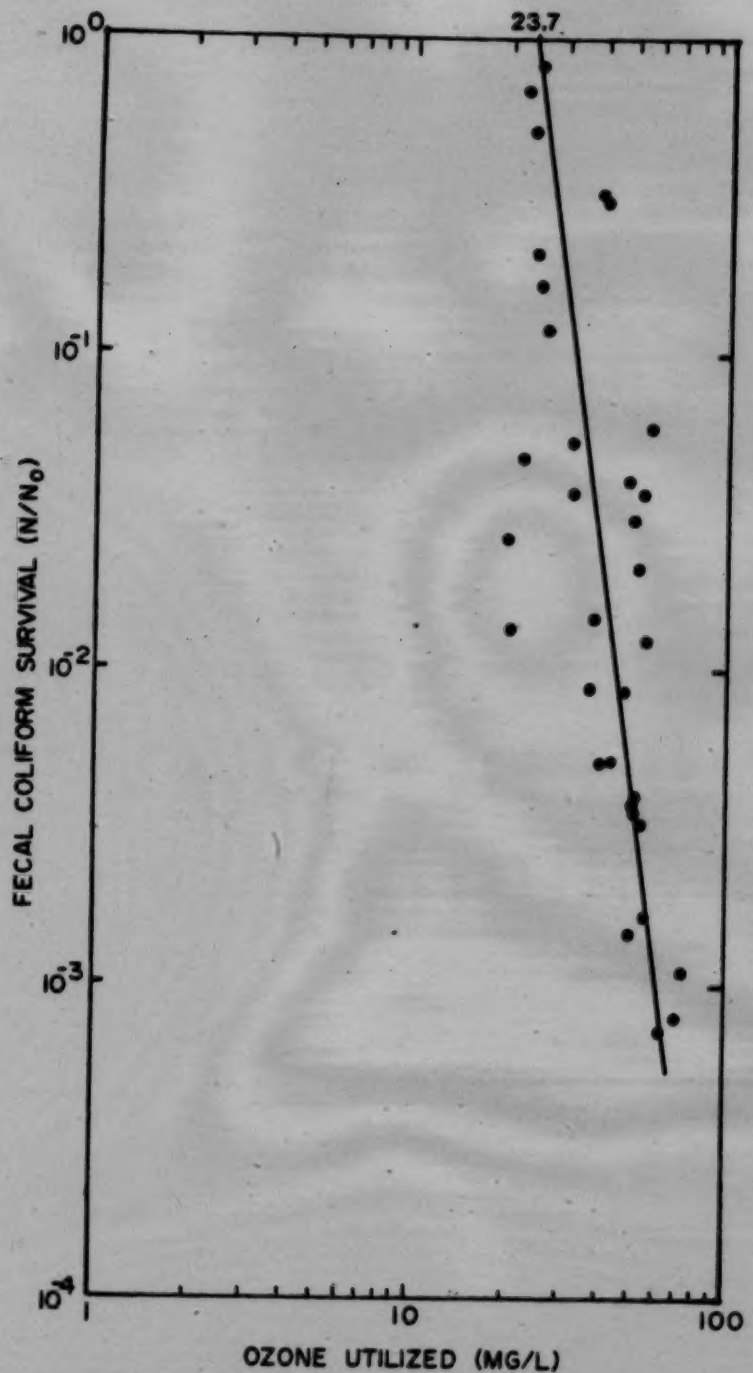

FIGURE 34: OZONE UTILIZED VS. FECAL COLIFORM SURVIVAL RATIO

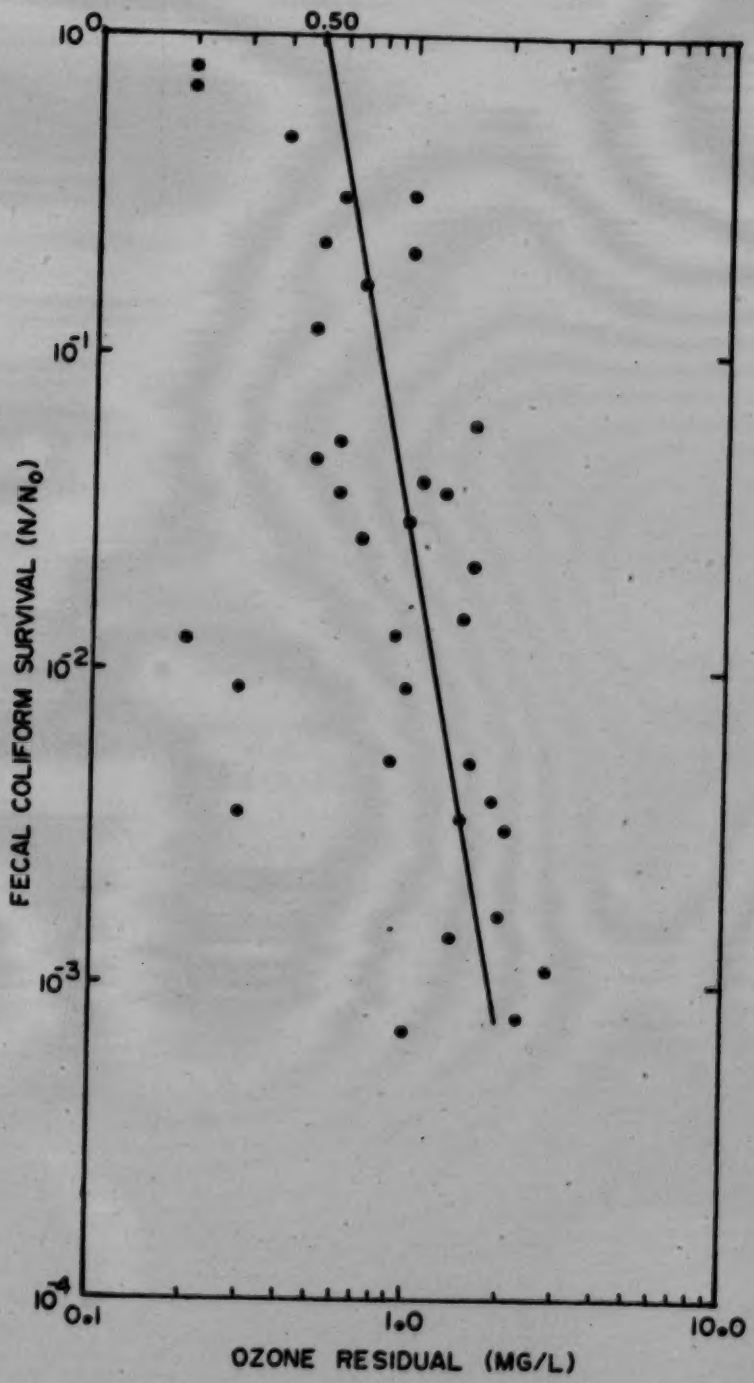

FIGURE 35: OZONE RESIDUAL VS. FECAL COLIFORM SURVIVAL RATIO

The values of the total and fecal coliform survival ratios (N/N_o) plotted in Figures 30 to 35 were obtained by averaging the six (6) total and fecal coliform counts obtained during each test (ie. one total coliform and one fecal coliform count for every retention period) to obtain an average count (N) of the total coliforms and fecal coliforms following disinfection. These averages were divided by the initial average (ie. non-disinfected average) of the total and fecal coliforms counts (N_o) obtained from the three (3) non-disinfected bacteriological samples to obtain the survival ratios (N/N_o). Average values were used because the counts of total and fecal coliforms present in each disinfected sample collected after each retention period were always within half an order of magnitude. Personal communication with J. Bell (1982) confirmed that numbers this closely associated should be considered to have the same meaning when dealing with coliform counts. As a result it was possible to average numbers together to obtain values for N and N_o.

Straight line linear regression equations developed for Figures 30 to 35 indicate that a log-log relationship exists for the plots. The intercepts of the regression lines with the abscissa range from 0.43 mg/L to 24.9 mg/L. These intercepts along with the slopes of the regression lines can be used to relate total and fecal coliform survival ratios to the applied ozone dose (D), ozone utilized (U) and ozone residual (R) in terms of equation 10 (Given and Smith, 1982).

$$\frac{N}{N_o} = \left(\frac{C}{a}\right)^b \qquad (10)$$

N_o - initial number of total coliforms or fecal coliforms per 100 ml

N - number of total coliforms or fecal coliforms surviving ozonation per 100 ml

$\frac{N}{N_o}$ - total or fecal coliform survival ratio

C - the applied ozone dose, or ozone utilized or ozone residual (mg/L)

a - the intercept with the abscissa, (mg/L)

b - slope of the regression line

The linear regression equations relating survival ratios to the applied ozone dose, ozone utilized or ozone residual are listed in Table 13.

D. Statistical Analysis

A stepwise multiple linear regression technique was used to determine the statistical significance between total and fecal coliform survival ratios (N/N_o) as the dependent variable and various possible independent variables. Numerous combinations of logarithmic (base 10) and non-logarithmic variables were examined to determine which provided the best predictive relationships for coliform survival.

Multiple linear regression equations involving logarithmically transformed values of applied ozone dose, ozone residual and primary effluent 5-day BOD provided the best correlation with log (N/N_o) for both total and fecal coliforms. The applied ozone dose may be replaced by the ozone utilized to provide an equation with only a slightly lower correlation. Table 14 presents a summary of the development of these equations and their corresponding R^2 values. The R^2 value is the square of the multiple correlation coefficient and it indicates the amount of the total variance in log (N/N_o) which may be accounted for by its regression on the independent variables.

The equations listed in Table 14 provide insight into the factors affecting the ozone disinfection of primary effluent; however due to the presence of 5-day BOD as an independent variable they are of little use for on-line operation of ozone disinfection systems using complete mix reactors. To compensate for this problem a data analysis was performed using only those variables which could be

TABLE 13 SIMPLE LINEAR REGRESSION EQUATIONS RELATING SURVIVAL RATIOS TO APPLIED OZONE DOSE, OZONE UTILIZED AND OZONE RESIDUAL

Total Coliforms		Fecal Coliforms	
$\frac{N}{N_o} = \left(\frac{C_d}{22.0}\right)^{-5.8}$,	r = 0.76 n = 33	$\frac{N}{N_o} = \left(\frac{C_d}{24.9}\right)^{-7.5}$,	r = 0.70 n = 33
$\frac{N}{N_o} = \left(\frac{C_u}{20.9}\right)^{-5.8}$,	r = 0.75 n = 33	$\frac{N}{N_o} = \left(\frac{C_u}{23.7}\right)^{-7.5}$,	r = 0.70 n = 33
$\frac{N}{N_o} = \left(\frac{C_r}{0.43}\right)^{-4.2}$,	r = 0.73 n = 28	$\frac{N}{N_o} = \left(\frac{C_r}{0.50}\right)^{-5.3}$,	r = 0.71 n = 28

$\frac{N}{N_o}$ - total or fecal coliform survival ratio
C_d - applied ozone dose, mg/L
C_u - ozone utilized, mg/L
C_r - ozone residual, mg/L
r - correlation coefficient
n - number of data points

TABLE 14 SUMMARY OF REGRESSION ANALYSIS FOR BACTERIAL SURVIVAL IN PRIMARY EFFLUENT

Organisms	Dependent Variable	Step	Regression Coefficients for b	log D	log U	log R	log BOD_5	R^2	n
Total Coliforms	$\log_{10} \frac{N}{N_o}$	1	3.70	-3.28				0.568	33
		2	-4.69	-3.46			4.20	0.808	33
		3	-5.22	-2.96		-0.44	4.05	0.833	33
Total Coliforms	$\log_{10} \frac{N}{N_o}$	1	3.58		-3.25			0.560	33
		2	-4.74		-3.41		4.15	0.795	33
		3	-5.30		-2.88	-0.48	4.00	0.824	33
Fecal Coliforms	$\log_{10} \frac{N}{N_o}$	1	4.28	-3.69				0.491	33
		2	-7.91	-3.95			6.10	0.837	33
		3	-8.44	-3.43		-0.46	5.95	0.855	33
Fecal Coliforms	$\log_{10} \frac{N}{N_o}$	1	4.15		-3.66			0.485	33
		2	-7.95		-3.90		6.05	0.825	33
		3	-8.54		-3.34	-0.50	5.88	0.847	33

Regression equation (with one to three independent variables):

$$\log \frac{N}{N_o} = b + b_1 \log X_1 + b_2 \log X_2 + b_3 \log X_3$$

$\frac{N}{N_o}$ - survival ratio for total and fecal coliforms

D - applied ozone dose, mg/L U = ozone utilized, mg/L R = ozone residual, mg/L

monitored by an operator on-line. The log (base 10) of the total and fecal coliform survival ratios were selected as dependent variables. Logarithmic transformations (base 10) of the ozone utilized, ozone residual, primary effluent turbidity, primary effluent temperature, disinfection retention time, primary effluent flowrate, ozonized carrier gas flowrate and ozone concentration in the carrier gas were used as independent variables. Logarithmic transformations of the ozone utilized, ozone residual and primary effluent turbidity were found to be the best independent variables for use in predictive relationships for on-line use. These equations are summarized in Table 15.

A multiple linear regression analysis was conducted to determine what factors affect: the amount of ozone utilized by a primary effluent and; the magnitude of an ozone residual established in a primary effluent. Primary effluent characteristics, operating characteristics and the applied ozone dose were used as independent variables. The results of this analysis are contained in Table 16.

TABLE 15 SUMMARY OF PREDICTIVE RELATIONSHIPS FOR ON-LINE USE

Organisms	Dependent Variable	Step	Regression Coefficients for b	log U	log R	log Turb	R^2	n
Total Coliforms	$\log_{10} \frac{N}{N_o}$	1	3.56	-3.24			0.555	32
		2	0.18	-3.80		2.75	0.622	32
		3	-2.62	-2.80	-0.98	3.50	0.724	32
Fecal Coliforms	$\log_{10} \frac{N}{N_o}$	1	4.09	-3.61			0.478	32
		2	-1.43	-4.53		4.50	0.602	32
		3	-5.07	-3.24	-1.27	5.47	0.721	32

Regression equation (with one to three independent variables):

$$\log \frac{N}{N_o} = b + b_1 \log X_1 + b_2 \log X_2 + b_3 \log X_3$$

$\frac{N}{N_o}$ - survival ratio for total and fecal coliforms

U - ozone utilized, mg/L R - ozone residual, mg/L

Turb - primary effluent turbidity, NTU

TABLE 16 MULTIPLE LINEAR REGRESSION EQUATIONS WITH OZONE RESIDUAL AND OZONE UTILIZED AS DEPENDENT VARIABLES FOR A COMPLETE MIX REACTOR SYSTEM

$\text{Log } R = 10.149 + 0.0131\ D - 0.0967\ \text{Temp} - 4.038\ \text{Log Alk}$	$R^2 = 0.572$
$\text{Log } U = -0.0272 - 0.0045\ \text{Temp} + 0.0072\ \text{pH} + 1.007\ \text{Log } D$	$R^2 = 0.998$

R - ozone residual, mg/L
U - ozone utilized, mg/L
D - applied ozone dose, mg/L
Temp - primary effluent temperature, °C
Alk - primary effluent alkalinity, mg/L $CaCO_3$
pH - primary effluent pH

VI. DISCUSSION

A. Dye Tests

Methylene blue dye is an organic dye. Its incomplete recovery may have been caused by its oxidation or absorbance. Since chlorine is an oxidizing agent, the test water (ie. sump water) was examined for it prior to the conduction of the dye tests. The chlorine concentration of the test water was found to be insignificant (<0.1 mg/L) when measured using a Penwalt Wallace & Tiernan amperometric titrator.

The low chlorine concentration in the test water makes it probable that the incomplete dye recovery was caused by adsorbance. The adsorbance may have occurred onto colloidal matter present in the test water during the dye tests and/or onto the plastic sample bottles. Since the test water was very clear and non-turbid the sample bottles presented the most likely adsorbing surfaces. Unfortunately tests were not conducted to determine if dye was actually lost by adsorbance onto the sample bottles. However, personal communication with G. Putz (1982) confirmed that rhodamine dye is adsorbed onto the surface of the plastic sample bottles used. It is quite possible that the methylene blue dye was lost in a similar manner.

B. Primary Effluent Characteristics

Table 17 lists a range of values for microbiological, physical and chemical characteristics of the primary effluent disinfected in this study. A comparison of these values with the general characteristics of primary effluents and raw wastewater listed in Table 10 indicate that the primary effluent used in this study had characteristics similar to those found in typical primary effluents.

TABLE 17 RANGE OF PRIMARY EFFLUENT CHARACTERISTICS IN THIS STUDY (taken from Appendix 2)

Characteristic	Range
5-day BOD (mg/L)	75.4 - 166.4
Suspended Solids (mg/L)	18.0 - 200.0
pH	7.30 - 8.80
Alkalinity (mg/L $CaCO_3$)	190.0 - 240.0
Total Coliforms (count/100 ml)	2.7×10^6 - 2.4×10^7
Fecal Coliforms (count/100 ml)	5.5×10^5 - 2.8×10^6

C. Control Tests

The coliform group of bacteria consist of aerobic and facultatively anaerobic bacilli (Pelczar and Reid, 1972) which should not be inactivated by oxygen. However control tests conducted in this study indicate that oxygen reduces the survival ratio of total and fecal coliforms present in primary effluent. The significance of the reduction in survival ratios can be determined using the student t test. When this test was applied the effect of oxygen on coliform survival ratios was found to be insignificant at the 1% probability level.

During each control test large volumes of froth/foam were formed on the surface of the primary effluent. It is possible that the reduction in total and fecal coliform survival ratios were caused by the removal of coliforms encapsulated in suspended solids which were removed by oxygen flotation. This method of suspended solids removal has been confirmed by Conway and Ross (1980) who report that suspended solids which neither settle nor rise at a significant rate such as those in primary effluent can be removed by dissolved air flotation. It is possible that microorganisms contained in the froth/foam formed during ozone disinfection are not inactivated due to protection given them by encapsulating solid matter. As a result the disposal of froth/foam formed during ozone disinfection may create bacterial and viral health hazards unless it is properly treated.

D. Disinfection Tests and Predictive Relationships

Large amounts of froth/foam formed during ozonation. The froth/foam was found to discharge through the carrier gas exhaust vent. As a result, it limits the liquid depth at which the reactor may be operated unless a froth/foam removal system is employed.

During each disinfection test, visual observations indicated that short circuiting of ozonized gas occurred. Although it was not possible to directly measure the amount of gas which short circuited the reactor, a comparison of the volume of gas leaving through the exhaust vent of the reactor (as measured by the wet test meter) with the amount entering the reactor (as estimated by using the rotameter on the ozone generator) indicated that less than two percent (2%) of the ozonized gas short circuited the reactor.

The intercepts of the regression lines with the abscissae in Figures 30 to 35 may be considered as the initial ozone demand of the primary effluent. The initial ozone demand is the amount of ozone which must be applied to, utilized by or present in the wastewater before any significant total and fecal coliform microorganism inactivation starts to occur. Once the initial ozone demand is satisfied not all additional applied ozone is employed in inactivating microorganisms. Some is required to meet the remaining ozone demand of the wastewater. As a result, the higher the wastewater quality the less ozone required to attain a given level of microorganism inactivation. This may be confirmed by comparing the applied ozone dose required to achieve disinfection for waters of various qualities as listed in Table 9.

Figures 30 to 35 exhibit a scatter of data points. This is expected because the concentration of ozone demanding substances in the primary effluent varied with each test. The greater the total ozone demand of a wastewater the less microorganism invactivation is attained for a given applied ozone dose once the initial ozone demand is satisfied. This means that equivalent microorganism inactivation is attainable in two wastewaters for the same applied ozone dose, ozone utilized or ozone residual only if the wastewaters have the same initial and

total ozone demand and if the ozone demanding substances are oxidized at the same rate by ozone.

The simple linear regression equations listed in Table 13 (which relate coliform survival ratios to applied ozone dose, ozone utilized and ozone residual) indicate that the equations for fecal coliforms have larger initial ozone demands (i.e. intercepts with the abscissae) and slopes of greater magnitudes than the equations for total coliforms. The intercepts and slopes of the fecal coliform equations were compared to those of the total coliform equations using the student t test. The differences between the intercepts were found to be insignificant at the 5% probability level, while the differences between the slopes were found to be significant at the 5% probability level. The difference between the slopes of corresponding fecal and total coliform equations is probably significant. The higher initial ozone demand for fecal coliforms inactivation may have been caused by the presence of ozone demanding substances in the primary effluent which were more easily oxidized than fecal coliforms but not total coliforms. This is possible, since the total coliform group consists of a wide variety of microorganisms some of which may be more easily oxidized than fecal coliforms. Once the initial ozone demand is satisfied the slopes of the simple linear regression equations indicate that additional ozone (in terms of applied ozone dose, ozone utilized or ozone residual) provides a greater degree of inactivation of fecal coliforms than total coliforms. This indicates that some members of the total coliform group of microorganisms may be more resistant to ozone oxidation than are fecal coliforms.

Table 18 compares simple linear regression equations relating coliform survival ratios to ozone utilized for a porous plate diffuser contacting system developed by Given and Smith (1982) to those for the complete mix reactor system developed in this study. The porous plate diffuser system consisted of two columns

TABLE 18 COMPARISON OF SIMPLE LINEAR REGRESSION EQUATIONS FOR COMPLETE MIX REACTOR AND POROUS PLATE DIFFUSER

Reactor	Wastewater	Total Coliforms	Fecal Coliforms
Complete Mix Reactor	Primary	$\frac{N}{N_o} = \left(\frac{C_u}{20.9}\right)^{-5.8}$	$\frac{N}{N_o} = \left(\frac{C_u}{23.7}\right)^{-7.5}$
Porous Plate Diffuser Reactor	Screened	$\frac{N}{N_o} = \left(\frac{C_u}{2.1}\right)^{-2.6}$	$\frac{N}{N_o} = \left(\frac{C_u}{1.8}\right)^{-2.9}$
Porous Plate Diffuser Reactor	RBC		$\frac{N}{N_o} = \left(\frac{C_u}{0.7}\right)^{-3.9}$
Porous Plate Diffuser Reactor	Lagoon		$\frac{N}{N_o} = \left(\frac{C_u}{3.4}\right)^{-4.6}$
Porous Plate Diffuser Reactor	Lagoon-septic tank waste		$\frac{N}{N_o} = \left(\frac{C_u}{12.5}\right)^{-3.1}$

with countercurrent flow of ozone gas. The initial ozone demand for the porous plate diffuser system was less than the initial ozone demand of the complete mix reactor system. This occurs because Given and Smith (1982) developed these equations while disinfecting dilute low strength wastewaters with a low initial ozone demand in comparison to the primary effluent examined in this study. The magnitude of the slopes of the equations for the complete mix reactor were approximately 2-3 times greater than the slopes of the equations for the porous plate diffuser system. The difference in slopes indicates that for equivalent wastewaters the complete mix reactor will achieve considerably greater microorganism inactivation for equivalent quantities of utilized ozone than the porous plate diffuser system.

Multiple linear regression equations presented in Tables 14 to 16 were developed to determine which operating characteristics and primary effluent characteristics have the greatest influence on the survival of total and fecal coliforms. From Table 14 it may be observed that log transformed values of the applied ozone dose, ozone residual and primary effluent 5-day BOD had the greatest effect on total and fecal coliform survival ratios. The applied ozone dose may be replaced by the ozone utilized with only a slight drop in the value of R^2. The equations of Table 14 indicate that a greater degree of fecal coliform inactivation is attained than total coliform inactivation for an equivalent applied ozone dose or amount of ozone utilized. This may be caused by some members of the total coliform group of organisms being more resistant to ozone oxidation than are fecal coliforms.

Table 19 compares the equations of Table 14 involving ozone utilized to those developed by Given and Smith (1982) for a porous plate diffuser system. From Table 19 it may be observed that wastewater temperature had a direct

TABLE 19 COMPARISON OF MULTIPLE LINEAR REGRESSION EQUATIONS FOR COMPLETE MIX AND POROUS PLATE DIFFUSER SYSTEMS

Organisms	Complete Mix Reactor	Porous Plate Diffuser System
Total Coliforms	$\log \frac{N}{N_o} = -5.30 - 2.88 \log(U) + 4.00 \log(BOD_5) - 0.48 \log(R)$	$\log \frac{N}{N_o} = -2.80 - 2.50 \log(U) + 1.3 \log(BOD_5) + 1.10 \log(T)$
	$R^2 = 0.824$	$R^2 = 0.742$
Fecal Coliforms	$\log \frac{N}{N_o} = -8.54 - 3.34 \log(U) + 5.88 \log(BOD_5) - 0.50 \log(R)$	$\log \frac{N}{N_o} = -3.80 - 2.50 \log(U) + 0.86 \log(T) - 0.15 \log(R) + 1.70 \log(BOD_5)$
	$R^2 = 0.847$	$R^2 = 0.875$

U - ozone utilized, mg/L

R - ozone residual, mg/L

BOD_5 - 5-day BOD, mg/L

T - wastewater temperature, °C

significant effect on the inactivation of total and fecal coliforms in a porous plate diffuser system. However due to the limited temperature range tested with the complete mix reactor system (13-16°C for the complete mix reactor vs. 5-16°C for the porous plate diffuser system) temperature could not be identified as a significant factor affecting coliform inactivation in complete mix reactor systems. When comparing the equations of Table 19 it is apparent that greater fecal and total coliform inactivation will occur in wastewaters disinfected in the complete mix reactor for equivalent amounts of ozone utilized and primary effluent 5-day BOD.

The predictive equations developed for on-line use by system operators for the complete mix reactor system (Table 15) were found to differ slightly from those developed for the porous plate diffuser system by Given and Smith (1982). The predictive equations for the two systems are compared in Table 20. The dependent variable in the porous plate diffuser system was the final fecal coliform count whereas in the complete mix reactor system the fecal coliform survival ratios were used. The predictive relationship developed by Given and Smith (1982) had four (4) independent variables whereas those developed for the complete mix reactor system had only three (3). This occurred because wastewater temperature was an important independent variable in the porous plate diffuser system but not in the complete mix reactor system for the experimental system used.

The equations presented in Table 16 may be used to explain some of the differences in the multiple linear regression relationships developed for the complete mix reactor and porous plate diffuser system. These equations show that the ozone residual formed in a wastewater in a complete mix reactor is directly

TABLE 20 COMPARISON OF PREDICTIVE RELATIONSHIPS FOR THE COMPLETE MIX AND POROUS PLATE DIFFUSER SYSTEMS

Reactor System	Fecal Coliforms
Complete Mix Reactors	$\log \frac{N}{N_o} = -5.07 - 3.24 \log(U) + 5.27 \log(Turb) - 1.27 \log(R)$
	$R^2 = 0.721$
Porous Plate Diffuser System	$\log \frac{N}{N_o} = 3.2 - 2.5 \log(U) + 1.5 \log(T) + 1.10 \log(Turb) - 0.20 \log(R)$
	$R^2 = 0.888$

U	-	ozone utilized, mg/L
R	-	ozone residual, mg/L
Turb	-	wastewater turbidity, NTU
T	-	wastewater temperature, °C
$\frac{N}{N_o}$	-	survival ratio of fecal or total coliforms
N	-	final count of total or fecal coliforms, (counts/100 ml).

affected by the wastewater temperature. Hence, although in this study wastewater temperature was not shown to have a direct effect on coliform survival ratio in a complete mix reactor it does have an indirect effect. Alkalinity and pH are also two wastewater characteristics which have a direct effect on the ozone residual formed in a wastewater. The effect of wastewater characteristics on the quantity of ozone utilized by a wastewater is small compared to the effect of the applied ozone dosage. As a result these wastewater characteristics also have an indirect effect on the inactivation of total and fecal coliforms by consuming residual ozone which may have otherwise been used to inactivate microorganisms.

VII. CONCLUSIONS

Based on the results of this study the following conclusions are applicable to a complete mix reactor system utilizing ozone as a wastewater disinfectant:

1) For comparable wastewaters and ozone utilized, complete mix reactors will achieve considerably greater coliform inactivation than the porous plate diffuser system examined by Given and Smith (1982). This may be due to the ability of complete mix reactors to: provide more rapid and efficient ozone mass transfer from the gas to the liquid phase; and to ensure complete dispersal of ozone bubbles throughout the liquid. The two reactor systems may be compared by using the simple and multiple linear regression equations listed in Tables 18 and 19.

2) BOD_5 is the wastewater characteristic which showed the greatest effect of those tested on the ozone inactivation of coliforms for the conditions tested. Wastewater characteristics such as temperature, pH, and alkalinity had a direct effect on the ozone residual maintained in a wastewater and hence had an indirect effect on the inactivation of coliforms.

3) The efficiency of disinfection of settled domestic wastewaters using a complete mix reactor may be predicted using a simple linear regression equation of the form:

$$\frac{N}{N_o} = \left(\frac{C}{a}\right)^b$$

$\frac{N}{N_o}$ = coliform survival ratio

C = applied ozone dose, ozone utilized, or ozone residual, (mg/L)

b = slope of the regression line

a = intercept with the abscissa, (mg/L)

For the system tested the constants are:

	Total Coliforms	Fecal Coliforms
C = applied ozone dose	a = 22.0 b = -5.8	a = 24.9 b = -7.5
C = ozone utilized	a = 20.9 b = -5.8	a = 23.7 b = -7.5
C = ozone residual	a = 0.43 b = -4.2	a = 0.50 b = -5.3

4) The efficiency of disinfection of settled domestic wastewater using a complete mix reactor can be predicted using a multiple linear regression equation of the form:

$$\log \frac{N}{N_o} = b + b_1 \log D + b_2 \log BOD_5 + b_3 \log R$$

$\frac{N}{N_o}$ = coliform survival ratio

D = applied ozone dose, (mg/L)

BOD_5 = 5-day biochemical oxygen demand, (mg/L)

R = Ozone residual, (mg/L)

For the system tested the coefficients are:

Total Coliforms	Fecal Coliforms
b = -5.22	b = -8.44
b_1 = -2.96	b_1 = -3.43
b_2 = 4.05	b_2 = 5.95
b_3 = -0.44	b_3 = -0.46

VIII. RECOMMENDATIONS

In order to further understand and improve the operation of a complete mix reactor for ozone disinfection of wastewaters the following factors are recommended for examination:

1) An adjustable overflow weir should be built into the reactor in order to simplify the adjustment of the liquid depth in the reactor. Such a level control device will eliminate the short circuiting of small amounts of ozonized gas which occurred during each test.
2) Tests should be conducted on a combined system of a porous plate diffuser and complete mix reactor to determine if the advantages of both systems can be combined to increase microorganism inactivation for a given applied ozone dose.
3) Studies should be conducted on the froth/foam formed during ozonation to determine if it presents a microbial health hazard.
4) Studies should be conducted to determine if parameters such as TOC or TOD may be effectively used to replace BOD_5. Such parameters could be rapidly measured on-line which would allow for more effective system control.

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APPENDIX 1

DYE TEST DATA

Dye Test Data

Theoretical dye concentrations for Figures 6 to 29 were calculated using equation 6.

$C_T = C_o e^{-t/\theta}$

C_T = dye concentration at time t, mg/L

C_o = inital dye concentration, M/V, mg/L

M = mass of dye injected, mg

V = reactor volume, L

t = time

θ = nominal detention time

Q = liquid flowrate, L/min

Data for Figures 6, 7, 8:

Operating depth = 375 mm.

θ = 3 min

V = 78 L

Q = 26 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.256	-	-	-
5	0.249	0.289	0.263	0.120
10	0.243	0.255	0.229	0.218
15	0.236	0.225	0.222	0.254
20	0.229	0.220	0.211	0.240
25	0.223	0.211	0.203	0.227
30	0.217	0.200	0.200	0.218
35	0.211	0.207	0.192	0.207
40	0.205	0.187	0.181	0.203
45	0.199	0.181	0.178	0.196
50	0.194	0.178	0.177	0.192
55	0.189	0.173	0.160	0.184
60	0.184	0.148	0.160	0.172
70	0.174	0.157	0.148	0.172
80	0.164	0.152	0.149	0.150
90	0.155	0.139	0.130	0.141
100	0.147	0.125	0.118	0.137
120	0.132	0.102	0.109	0.127
150	0.111	0.075	0.083	0.085
180	0.094	0.060	0.057	0.068
210	0.080	0.039	0.048	0.050
240	0.068	0.031	0.035	0.046

Data for Figures 9, 10, 11:

Operating Depth = 375 mm.

θ = 5 min.

V = 80 L

Q = 16 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.250	-	-	-
5	0.246	0.120	0.175	0.210
10	0.242	0.290	0.250	0.240
15	0.238	0.340	0.234	0.230
20	0.234	0.267	0.223	0.228
25	0.230	0.208	0.215	0.215
30	0.226	0.202	0.215	0.215
35	0.222	0.202	0.207	0.213
40	0.219	0.190	0.207	0.210
45	0.215	0.180	0.207	0.205
50	0.212	0.178	0.203	0.205
55	0.208	0.170	0.197	0.197
60	0.205	0.170	0.193	0.195
70	0.198	0.160	0.186	0.188
80	0.191	0.160	0.172	0.170
90	0.185	0.142	0.168	0.165
100	0.179	0.142	0.158	0.158
120	0.168	0.121	0.140	0.144
150	0.152	0.105	0.120	0.124
180	0.137	0.094	0.094	0.095
210	0.124	0.078	0.085	0.076
240	0.112	0.062	0.073	0.062
300	0.092	0.062	0.066	0.050
360	0.075	0.050	0.050	0.040

Data for Figures 12, 13, 14:

Operating Depth = 375 mm

θ = 7 min.

V = 77 L

Q = 11 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.260	-	-	-
5	0.257	0.085	0.170	0.211
10	0.254	0.300	0.271	0.244
15	0.251	0.340	0.243	0.236
20	0.248	0.247	0.235	0.226
25	0.245	0.213	0.227	0.222
30	0.242	0.205	0.227	0.218
35	0.239	0.197	0.214	0.214
40	0.236	0.205	0.210	0.202
45	0.233	0.199	0.207	0.202
50	0.231	0.195	0.202	0.191
55	0.228	0.195	0.193	0.191
60	0.225	0.188	0.190	0.188
70	0.220	0.182	0.185	0.176
80	0.215	0.175	0.177	0.169
90	0.210	0.164	0.172	0.167
100	0.205	0.164	0.159	0.167
120	0.195	0.148	0.141	0.161
150	0.182	0.130	0.121	0.141
180	0.169	0.117	0.094	0.124
210	0.158	0.107	0.094	0.103
240	0.147	0.093	0.072	0.085
300	0.127	0.062	0.063	0.068
360	0.110	0.042	0.055	0.062
420	0.096	0.033	0.035	0.045
480	0.083	0.023	0.024	0.032

Data for Figures 15, 16, 17:

Operating Depth = 500 mm

θ = 3 min.

V = 102 L

Q = 34 L/min

M = 20 mg/L

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.196	-	-	-
5	0.191	0.231	0.090	0.203
10	0.185	0.187	0.206	0.177
15	0.180	0.177	0.206	0.174
20	0.175	0.173	0.196	0.174
25	0.171	0.167	0.190	0.166
30	0.166	0.161	0.180	0.166
35	0.161	0.161	0.185	0.162
40	0.157	0.156	0.174	0.154
45	0.153	0.152	0.170	0.145
50	0.149	0.142	0.172	0.132
55	0.144	0.139	0.170	0.132
60	0.140	0.135	0.146	0.127
70	0.133	0.122	0.123	0.123
80	0.126	0.108	0.133	0.115
90	0.119	0.093	0.120	0.110
100	0.113	0.088	0.106	0.090
120	0.101	0.075	0.085	0.080
150	0.085	0.061	0.068	0.068
180	0.072	0.052	0.056	0.050
210	0.061	0.039	0.044	0.042
240	0.052	0.033	0.034	0.034

Data for Figures 18, 19, 20:

Operating Depth = 500 mm

θ = 5 min.

V = 100 L

Q = 20 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.200	-	-	-
5	0.197	0.155	0.152	0.211
10	0.193	0.226	0.182	0.189
15	0.190	0.243	0.165	0.189
20	0.187	0.176	0.165	0.184
25	0.184	0.176	0.154	0.180
30	0.181	0.173	0.152	0.178
35	0.178	0.159	0.148	0.178
40	0.175	0.159	0.143	0.162
45	0.172	0.155	0.139	0.160
50	0.169	0.155	0.139	0.155
55	0.167	0.151	0.122	0.154
60	0.164	0.146	0.139	0.151
70	0.158	0.137	0.122	0.138
80	0.153	0.129	0.122	0.128
90	0.148	0.121	0.118	0.126
100	0.143	0.116	0.113	0.119
120	0.134	0.107	0.103	0.100
150	0.121	0.090	0.087	0.096
180	0.110	0.069	0.070	0.072
210	0.099	0.055	0.056	0.065
240	0.090	0.051	0.051	0.047
300	0.074	0.033	0.030	0.037

Data for Figures 21, 22, 23:

Operating Depth = 500 mm

θ = 7 min.

V = 105 L

Q = 15 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.190	-	-	-
5	0.188	0.096	0.156	0.203
10	0.186	0.164	0.189	0.193
15	0.183	0.230	0.175	0.178
20	0.181	0.174	0.171	0.176
25	0.179	0.164	0.166	0.175
30	0.177	0.162	0.164	0.170
35	0.175	0.162	0.156	0.165
40	0.173	0.158	0.156	0.160
45	0.171	0.158	0.156	0.161
50	0.169	0.140	0.153	0.157
55	0.167	0.129	0.148	0.153
60	0.165	0.136	0.148	0.153
70	0.161	0.125	0.140	0.135
80	0.157	0.125	0.140	0.130
90	0.153	0.140	0.135	0.126
100	0.150	0.106	0.131	0.123
120	0.143	0.099	0.105	0.108
150	0.133	0.093	0.097	0.088
180	0.124	0.080	0.076	0.075
210	0.115	0.068	0.066	0.070
240	0.107	0.057	0.063	0.058
300	0.093	0.044	0.047	0.040
360	0.081	0.034	0.031	0.028

Data for Figures 24, 25, 26:

Operating Depth = 250 mm

θ = 3 min.

V = 51 L

Q = 17 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.392	-	-	-
5	0.381	-	0.442	0.357
10	0.371	0.502	0.317	0.315
15	0.361	0.407	0.317	0.302
20	0.351	0.315	0.305	0.287
25	0.341	0.275	0.292	0.275
30	0.332	0.268	0.275	0.261
35	0.323	0.258	0.262	0.252
40	0.314	-	0.255	0.244
45	0.305	0.242	0.245	0.237
50	0.297	0.223	0.232	0.227
55	0.289	0.221	0.221	0.219
60	0.281	0.218	0.225	0.215
70	0.266	0.213	0.203	0.194
80	0.251	0.190	0.197	0.181
90	0.238	0.172	0.180	0.167
100	0.225	0.165	0.162	0.151
120	0.201	0.130	0.133	0.133
150	0.170	0.090	0.097	0.095
180	0.144	0.075	0.079	0.077
210	0.122	-	0.064	0.057
240	0.103	0.040	0.047	-

Data for Figures 27, 28, 29:

Operating Depth = 250 mm

θ = 5 min.

V = 52.5 L

Q = 10.5 L/min

M = 20 mg

Time (sec)	Theoretical Dye Concentration (mg/L)	Measured Dye Concentrations 60 rpm	180 rpm	300 rpm
0	0.381	-	-	-
5	0.375	0.013	0.482	0.409
10	0.369	0.367	0.312	0.328
15	0.362	0.437	0.322	0.322
20	0.356	0.396	0.311	0.309
25	0.351	0.301	0.305	0.297
30	0.345	0.288	0.282	0.292
35	0.339	0.284	0.266	0.277
40	0.333	0.272	0.260	0.266
45	0.328	0.263	0.252	0.256
50	0.223	0.251	0.243	0.252
55	0.217	0.246	0.238	0.248
60	0.212	0.241	0.236	0.235
70	0.302	0.228	0.232	0.225
80	0.292	0.214	0.221	0.218
90	0.282	0.198	0.209	0.204
100	0.273	0.185	0.201	0.199
120	0.255	0.163	0.192	0.174
150	0.231	0.138	0.167	0.145
180	0.209	0.101	0.140	0.132
210	0.189	0.074	0.110	0.099
240	0.171	0.061	0.062	0.089
300	0.140	0.036	0.037	0.058
360	0.115	0.022	0.019	0.032

APPENDIX 2

DISINFECTION TEST DATA

Test No.	Liquid Depth (mm)	V (L)	T.S. (r.p.m.)	Q_L (L/min)	Q_G (L/min)	C_1 (mg/L)	C_2 (mg/L)	D (mg/L)	U (mg/L)	R (mg/L)
1	385	78	300	22	25.7	21.0	1.9	24.5	22.3	0.2
2	385	78	300	20	25.8	21.0	1.7	27.1	24.9	0.2
3	385	78	300	18	26.1	37.0	2.0	53.7	50.8	1.9
4	385	78	300	16	26.1	32.1	1.7	52.4	49.6	1.4
5	385	78	300	14	26.2	32.1	1.6	60.1	57.1	1.6
6	385	78	300	10	24.1	23.5	2.0	56.6	51.8	1.5
7	385	78	300	12	26.1	23.5	1.2	51.1	48.5	1.1
8	385	78	300	15	25.6	33.1	1.1	56.5	54.6	1.3
9	385	78	300	17	25.5	33.1	1.4	49.7	47.6	1.0
10	385	78	300	21	24.8	20.6	1.4	24.3	22.7	0.5
11	385	78	300	10	25.6	20.6	0.7	52.7	50.9	1.0
12	385	78	300	15	25.7	33.7	1.0	57.7	56.0	0.2

Test No	TC_o	TC_f	FC_o	FC_f	BOD_5 (mg/L)	S.S. (mg/L)	Temp. (°C)	pH	Turb. (NTU)	ALK.	R.T. (min)
1	2.4E7	1.1E7	1.8E6	1.2E6	144.0	58.7	16.0	8.10	34	225.0	3.55
2	1.4E7	8.3E6	9.4E5	7.5E5	158.7	46.0	16.0	8.30	28	215.0	3.90
3	1.1E7	8.3E4	2.0E6	7.7E3	113.2	96.7	14.0	7.30	36	210.0	4.33
4	7.0E6	2.7E4	1.4E6	2.0E3	88.6	67.3	14.0	8.40	29	225.0	4.88
5	5.3E6	3.1E5	2.8E6	1.6E5	148.0	94.0	14.0	7.80	43	230.0	5.57
6	1.1E7	5.1E4	1.8E6	6.1E3	142.4	70.0	13.0	7.70	38	200.0	7.80
7	9.7E6	5.4E4	2.5E6	9.7E4	154.4	75.0	14.5	7.85	43	220.0	6.50
8	1.7E7	6.0E5	1.9E6	6.7E4	133.6	62.0	14.0	8.00	43	235.0	5.20
9	1.4E7	1.2E5	1.4E6	1.2E4	113.6	52.0	14.0	7.80	34	220.0	4.59
10	7.9E6	5.2E5	1.5E6	6.8E4	113.6	50.0	15.0	8.40	28	230.0	3.71
11	8.1E6	4.6E5	2.3E6	6.6E4	166.4	76.0	15.0	8.50	43	240.0	7.80
12	1.1E7	1.9E5	9.9E5	1.2E4	102.0	88.0	14.0	8.40	34	235.0	5.20

Test No.	Liquid Depth (mm)	V (L)	T.S. (r.p.m.)	Q_L (L/min)	Q_G (L/min)	C_1 (mg/L)	C_2 (mg/L)	D (mg/L)	U (mg/L)	R (mg/L)
13	385	78	300	19	24.8	28.8	1.2	37.6	36.0	0.3
14	385	78	300	8	26.0	20.5	0.8	66.6	64.0	1.0
15	385	78	300	8	25.5	18.4	0.9	58.7	55.8	2.0
16	385	78	300	8	25.5	18.4	1.3	58.7	54.5	2.1
17	385	78	300	7	25.2	21.8	1.7	78.5	72.4	2.3
18	385	78	300	7	25.3	21.8	1.6	78.8	73.0	2.8
19	385	78	300	22	24.8	21.6	0.9	24.3	23.3	0.4
20	385	78	300	20	25.1	21.6	0.4	27.1	26.6	0.5
21	385	78	300	16	25.1	21.6	0.6	33.9	32.9	0.6
22	385	78	300	10	25.5	21.6	0.7	55.1	53.3	1.6
23	385	78	300	18	24.6	25.2	1.3	34.4	32.7	0.6
24	385	78	300	14	24.9	25.2	0.9	44.8	43.2	0.9

Test No	TC_o	TC_f	FC_o	FC_f	BOD_5 (mg/L)	S.S. (mg/L)	Temp. (°C)	pH	Turb. (NTU)	ALK.	R.T. (min)
13	6.9E6	1.0E5	7.5E5	6.5E3	93.2	40.0	15.0	8.50	31	225.0	4.11
14	7.6E6	1.8E4	7.2E5	5.1E2	91.2	18.0	14.0	8.75	26	230.0	9.75
15	9.3E6	2.0E4	9.6E5	1.6E3	95.3	72.0	14.3	8.30	36	195.0	9.75
16	1.0E7	5.1E4	1.7E6	5.3E3	117.4	65.0	14.0	8.40	43	215.0	9.75
17	1.5E7	2.0E4	9.4E5	7.4E2	103.7	N.A.	13.5	7.30	48	200.0	11.14
18	7.3E6	1.5E4	7.9E5	8.8E2	109.0	N.A.	14.0	7.45	45	200.0	11.14
19	1.5E7	3.9E6	1.1E6	5.2E5	119.8	52.0	13.8	8.30	39	220.0	3.55
20	9.7E6	7.6E5	7.9E5	9.2E4	113.6	44.0	13.8	8.35	33	210.0	3.90
21	6.9E6	3.0E5	8.3E5	2.9E4	109.2	78.0	13.8	8.75	31	220.0	4.88
22	9.1E6	2.4E5	1.5E6	3.1E4	138.4	92.0	13.8	8.60	43	230.0	7.80
23	1.2E7	4.8E5	9.7E5	4.9E4	126.4	68.0	14.0	7.90	42	205.0	4.33
24	8.6E6	6.5E4	6.4E5	3.3E3	116.0	48.0	14.3	8.15	34	195.0	5.57

Test No.	Liquid Depth (mm)	V (L)	T.S. (r.p.m.)	Q_L (L/min)	Q_G (L/min)	C_1 (mg/L)	C_2 (mg/L)	D (mg/L)	U (mg/L)	R (mg/L)
25	385	78	300	12	25.1	25.2	0.5	52.7	51.7	0.3
26	500	101	300	20	25.3	16.7	0.7	21.1	20.2	0.7
27	500	101	300	18	25.7	16.7	1.2	23.4	21.7	0.9
28	500	101	300	16	25.1	16.7	0.7	26.2	25.1	0.7
29	500	101	300	10	25.1	16.6	0.6	41.7	40.1	0.6
30	500	101	300	10	25.0	16.6	0.8	41.5	39.5	1.0
31	500	101	300	16	24.9	16.6	0.8	25.8	24.6	1.0
32	500	101	300	10	25.3	16.5	1.0	41.7	39.2	1.5
33	500	101	300	10	25.2	16.5	0.8	41.6	39.8	1.6

Test No	TC_o	TC_f	FC_o	FC_f	BOD_5 (mg/L)	S.S. (mg/L)	Temp. (°C)	pH	Turb. (NTU)	ALK.	R.T. (min)
25	6.7E6	5.3E4	6.2E5	2.2E3	100.0	64.0	14.5	8.80	N.A.	205.0	6.50
26	2.7E6	1.4E5	5.5E5	1.4E4	75.4	64.0	13.5	8.65	27	200.0	5.05
27	4.3E6	1.0E5	1.0E6	1.3E4	78.0	84.0	14.0	8.40	26	190.0	5.61
28	7.5E6	1.5E6	1.5E6	2.4E5	120.0	92.0	14.8	8.50	42	205.0	6.32
29	9.8E6	1.8E6	1.1E6	3.3E5	144.0	168.0	13.7	7.70	38	215.0	10.10
30	1.1E7	1.5E6	8.8E5	2.7E5	125.6	28.0	13.8	7.60	33	200.0	10.10
31	9.9E6	1.7E6	1.2E6	2.4E5	124.0	200.0	14.5	8.10	42	210.0	6.32
32	9.7E6	1.4E5	9.1E5	1.3E4	120.0	128.0	14.0	7.60	36	190.0	10.10
33	7.0E6	7.9E4	9.4E5	4.7E3	109.6	104.0	14.2	7.90	32	190.0	10.10

Liquid Depth - in reactor

V - volume of liquid in reactor

T.S. - Turbine speed

Q_L, Q_G - liquid and carrier gas flowrates, Q_G is in standard L/min

C_1,C_2 - concentration of ozone entering and leaving the reactor

D, U, R - applied ozone dose, ozone utilized, ozone residual

Turb - primary effluent turbidity

Alk -primary effluent alkalinity mg $CaCO_3$/L

R.T. - retention time

TC_o, TC_f FC_o, FC_f - initial and final total and fecal coliform counts per 100 ml